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The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Airport Paper No. 24

"The Effects of Jet Aircraft on Airfield Pavements, and Associated Problems" *

by

Douglas Hornsby Carrack, A.M.I.C.E., and Douglas Gordon Robertson, M.Eng., A.M.I.C.E.

SYNOPSIS

The Paper begins by outlining the problem, and indicating the first noticeable effects of jet engines and fuels on the existing types of airfield pavements. These effects are analysed and details are given of experiments carried out to determine the temperature characteristics of the jet stream and its action, under controlled conditions, upon various types of construction at present in common use for airfield pavements. The practical application of these tests is discussed and the results are presented in graphical form to enable the probable effect of any jet aircraft on existing types of construction to be assessed. Conclusions are drawn regarding the relative merits of the existing types of construction.

Arising from this the development of materials to resist the various effects of jet aircraft are discussed and details are given of laboratory work and field tests carried out.

A number of further problems associated with the use of jet-propelled aircraft are briefly mentioned.

In conclusion the present position is summarized and reference is made to the various ways by which aircraft designers and operators could assist in alleviating the present problems.

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Introduction

In recent years considerable advancement has been made in the development of design data, plant, and site-control methods to ensure the achievement of a high standard in the design and construction of airfield pavements, but the advent of the jet-propelled aircraft has brought with it new civil engineering problems.

The first flight of a British jet-propelled aeroplane took place in May 1941. The aircraft was a Gloster E.28/39—a product of the Gloster Aircraft Company, powered with a Whittle engine. The first twin-engined jet aircraft, the Meteor, flew in March 1943 and the first Royal Air Force squadron of Meteors, No. 616 Squadron, was formed at Manston in 1944. Meanwhile, in September 1943, the de Havilland Company's Vampire first flew experimentally. With the introduction of these types of aircraft into squadron service with the Royal Air Force there came an indication of the troubles that were likely to arise from the increasing use of airfields by jet-propelled aircraft, and this was first mentioned by C. E. Foster in 1949.[1] The inaugural flight of the de Havilland Comet, the first civil jet-propelled airliner put into service with the British Overseas Airways Corporation in 1952, led to the realization that any problems arising in respect of jet-propelled aircraft must also be borne in mind in the consideration of civil airfields.

Prior to the introduction of jet-propelled aircraft, the approach to the design of the runways, taxi-tracks, and servicing areas had been mainly confined to determining, from established criteria, the most economical form of construction, that is, rigid or flexible, based on the existing subgrade conditions and bearing in mind the nature and availability of local materials. It is now apparent that many special requirements and materials will have to be investigated by the airfield engineer to provide for the increasing use of both new and existing airfields by the present and future types of jet-propelled aircraft.

The first serious problems in Great Britain were, quite naturally, those encountered on airfields used by the Royal Air Force. It was originally thought that the main problems arising from the introduction of these aircraft would prove to be those resulting purely from the effects of the high temperatures and exhaust velocities from the jet engine and, in fact, a number of serious problems did arise almost immediately from these causes. In addition, whereas the fuels used in piston-engined aircraft are highly volatile and little trouble is caused by spillage, the fuels used by jet-propelled aircraft lie about, particularly on servicing areas, resulting in considerable softening of asphalt surfacings or joint-sealing compounds.

[1] The references are given on p. 29.

Existing Pavements

A survey of airfields from which jet aircraft have been operating indicates generally that high-grade concrete performs satisfactorily under normal conditions, whilst areas of asphalt and tarmacadam are damaged in varying degrees by these aircraft.

There have been instances of concrete being affected by the heat and blast from the jet engine. B. H. Knight [2] illustrated the type of damage which was done to the concrete at London Airport by a Viking Aircraft fitted with Nene engines for experimental purposes. This machine has a tailwheel and the angle of inclination of the jet is greater than is the case with aircraft which have tricycle undercarriages. The inclination of the jet was slightly in excess of 10 degrees, and the temperature of the exhaust gases would be about 350° C. on striking the pavement surface ; in addition it is understood that the run-up period was considerably longer than is normally to be expected. Spalling has been noted in the concrete around the joints ; invariably, the joints affected were formed by a steel bar and had been finished off with a trowel and builder's float, the concrete being worked up into a fatty state. Such areas are particularly susceptible to the effects of the jet stream and may be stripped off, leaving the aggregate exposed.

At a number of airfields which had been operating quite satisfactorily with piston-engined aircraft, difficulties arose when jet-propelled aircraft began to use the runways. The concrete was of wartime construction, about 6 to 7 inches in thickness, and had been surface-dressed a number of times with tar and grit. It was found that any areas of excessive surface laitance in the concrete under the tar dressing were particularly susceptible to the jet blast. Until this time it was felt that any such damage would be restricted to the ends of the runways and servicing areas ; however, it soon became apparent that where pilots were engaged in flying training and doing what is known as " circuits and bumps " there was a danger from the effects of blast on a poor-quality concrete where the pilot touched down and took off again immediately afterwards.

The main difficulty so far encountered on site in connexion with concrete pavements has been the provision of a satisfactory joint-sealing compound. The rubber/bitumen compounds which were in general use at the time of the introduction of jet aircraft were all affected in varying degrees by the new fuels used, and in addition were softened and quickly displaced when subjected to jet blast. *Fig. 1* gives an indication of the distance to which the blast from a Vampire can remove joint-sealing compounds from joints in concrete slabs. Spillage proved so serious on the servicing and run-up areas that the joint-sealing compound, particularly during the summer, approached a near-liquid condition and remained constantly tacky. The sealing compounds under these conditions present little or no resistance to jet blast, resulting in a sticky mess covering a

large area of the concrete. The tires of aircraft passing across these areas acquired a coating of the compound and, what was even worse, the bitumen on the surface of the concrete was invariably transferred from the flying boots of aircrew on to flying clothing, into the cockpits, and in some cases on to parachutes.

Various attempts to overcome the immediate site problem included painting the existing joint-sealing compounds with protective paints or treating with a cement slurry. Neither of these treatments was successful and trafficking with aircraft and heavy fuelling bowsers soon completely destroyed the protective coatings. As a temporary expedient, instructions were issued to keep the sealing compound down to $\frac{3}{8}$ inch below the surface of the concrete when pouring on new works. This minimized the effect on only those joints at right angles to the jet blast; where the blast of the jet was directly aligned with a joint, the sealing compound was softened by the heat of the blast and blown out of the joint opening for some distance behind the aircraft. In addition, the practice of underfilling the joint caused a channel into which the jet fuel flowed and remained, keeping the sealing compound in a softened state; the solvent action of the fuel caused an almost continual loss in the depth of effective seal.

Considerable areas of bituminous surfacings exist on airfields in Great Britain. Such areas are generally hot-mixed asphalt wearing-courses $\frac{3}{4}$ to 1 inch thick, either on tarmacadam base-course or as surfacing on existing concrete. The latter cannot be described as flexible pavements in the true sense but they are included here for convenience.

Asphalt carpets are very susceptible to blast from jet-propelled aircraft with low-slung engines. The first indications of damage are smears on the surface (see *Fig. 2*) where the binder is melted and blown away leaving the coarse aggregate exposed. Such damage is to be seen particularly at the runway ends and the adjacent taxi-tracks, the degree of damage varying directly in accordance with the intensity of traffic. Where a single wearing-course of asphalt exists on concrete, damage is invariably severe when traffic is intense, large areas of asphalt being blown away leaving the concrete exposed. *Fig. 4* gives an indication of such damage typical of maintenance areas.

Observations indicate that open-textured surfaces may be slightly more susceptible to heat and blast damage than smooth-textured surfaces; in addition, the fuels lie about in the interstices for longer periods. There is also an indication that asphalt's resistance to fuels increases with age. The fuel appears to soak into new pavements more easily than into the older pavements, thereby producing softening to a greater depth. This may be attributable to the fact that the older areas are denser as a result of trafficking. Fuelling, wherever possible, is carried out on concrete areas and fuel spillage on asphalt areas is generally confined to spillage either from the aircraft (from wet starts and shutting down the engine) or drippings from fuel bowsers. The damage from spillage is therefore

not generally extensive, but nevertheless it is noticeable on a detailed inspection and it is much more apparent on a hot summer day.

Fuels used by Jet Aircraft

The two main types of fuel used by jet-propelled aircraft in Great Britain are AVTUR and AVTAG. The solvent action of these fuels on bituminous and tarmacadam carpets is caused mainly by their aromatic content, since both bitumens and tars are soluble to varying degrees in aromatics; therefore, the higher the aromatic content the more virulent the fuel. AVTUR has an aromatic content ranging from 7 to 12 per cent and AVTAG has an aromatic content ranging from 15 to 25 per cent. It is understood that a reasonable and all-round economical figure for aromatic content is in the region of 20 per cent, and it may well be practicable to limit the aromatic content to this figure in the future, but meanwhile allowance must be made for the higher figure.

It is therefore apparent that such fuels coming into contact with bituminous and tarmacadam surfaces or bitumen-based joint-sealing compounds in concrete pavements will result in their deterioration, thereby considerably reducing the effective life of the materials.

General Problem

The immediate problems which arose on airfield pavements from the use of jet-propelled aircraft, as distinct from piston-engined aircraft, were those resulting directly from heat and blast from the engine and from fuel spillage occurring either from the servicing of the aircraft or from the starting and shutting down of the jet engine. It was obvious that the extent of the damage caused directly by the jet efflux would depend upon the location of the jet orifice in the aircraft and the direction of the jet stream relative to the pavement. Three types of aircraft at present in use are shown in *Figs 3*; these illustrations show clearly that the Attacker would cause more damage than either the Meteor or the Sabre. It was, however, necessary to establish a basis from which a fairly accurate assessment could be made of the damage likely to be caused on existing types of construction by the jet stream from any aircraft. The temperatures and exhaust velocities which may be said to be critical to airfield designers are those existing when they reach the pavement level and any materials used in the surface should be capable of resisting them. It was essential to find a satisfactory method of determining the temperature distribution of the exhaust blast impinging on a pavement for any size of jet pipe and orifice temperature, given its angle of inclination and height above the pavement level, and with this in mind a series of field tests was put in hand at Cardington.

Jet Efflux

Temperature and Velocity Characteristics

It is convenient to express the temperature at any point in the exhaust stream of a jet engine in the form of a parameter $\frac{T-T_o}{T_j-T_o}$ where T denotes

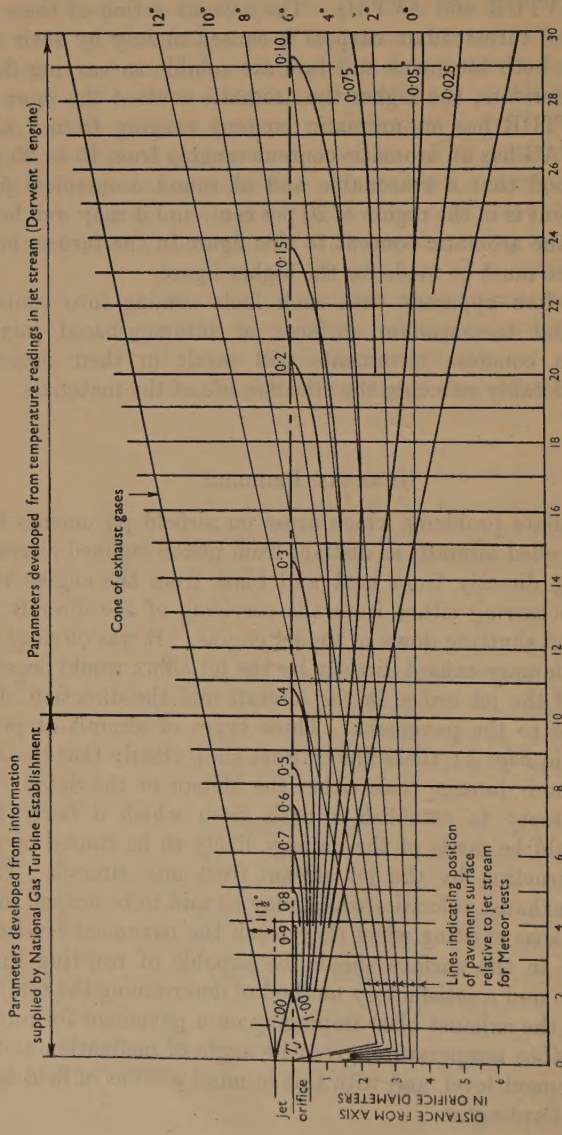


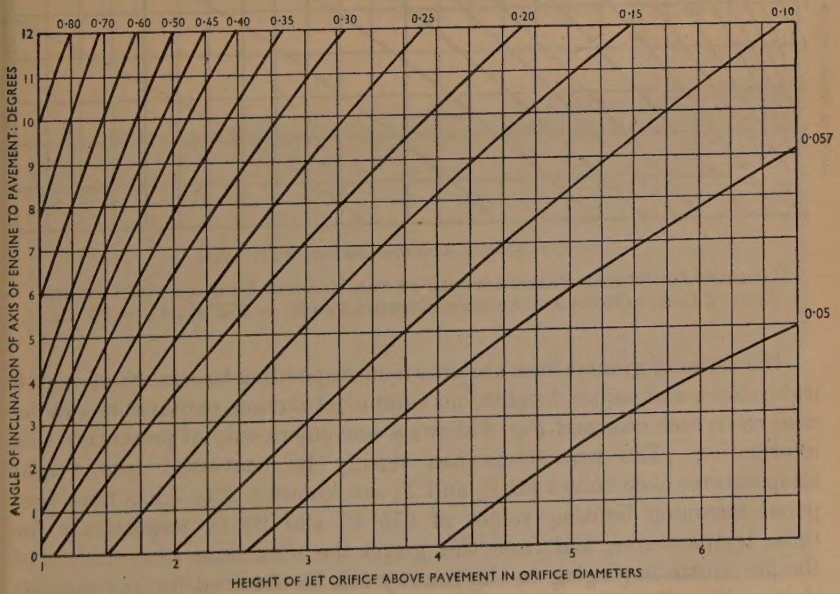
Fig. 5

Axial-Section of Jet Exhaust Stream showing Temperature Distribution in Terms of Parameter $\frac{T-T_o}{T_j-T_o}$

the temperature at the point considered, and the suffixes o and j refer to ambient and jet-orifice conditions respectively. The distance of the point from the jet orifice can then be defined in terms of orifice diameters, and a set of temperature curves can be drawn up for the jet stream on a non-dimensional basis which will apply with reasonable accuracy to any gas turbine engine of the hot-jet type.

The temperature distribution up to a distance of 10 diameters from the orifice was obtained from information supplied by the National Gas Turbine Establishment, and additional temperature measurements beyond this point were made by fixing thermocouples in the jet stream of a Derwent Mark I engine at Cardington. This enabled the temperature distribution curves shown in *Fig. 5* to be plotted to a distance of 30 diameters from the orifice. As shown in the diagram, the wake spreads at an angle of 11½ degrees to the axis of the jet stream. In a similar manner, velocities in the jet stream can be expressed in terms of the parameter $\frac{V-V_o}{V_j-V_o}$, and up to a distance of 10 diameters from the orifice the velocity distribution curves in parametric form are very similar to those for temperature distribution. The spread of the velocity wake is, however, rather less than that for the temperature, making an angle of 7½ degrees to the axis of the jet stream.

Fig. 6

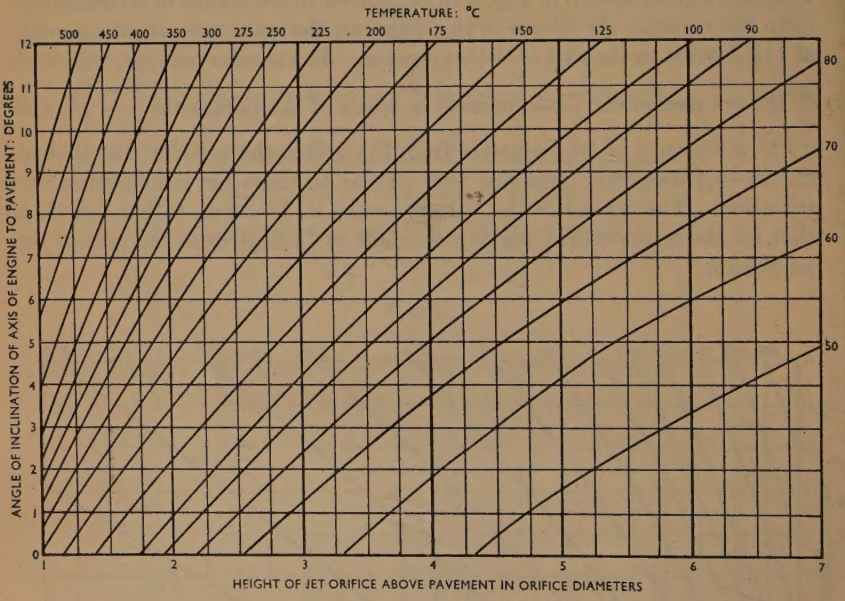

0·80 0·70 0·60 0·50 0·45 0·40 0·35 0·30 0·25 0·20 0·15 0·10
0·057
0·05
ANGLE OF INCLINATION OF AXIS OF ENGINE TO PAVEMENT: DEGREES
HEIGHT OF JET ORIFICE ABOVE PAVEMENT IN ORIFICE DIAMETERS

PARAMETERS SHOWING JET-STREAM TEMPERATURES AT THE RUNWAY SURFACE FOR VARIOUS ENGINE INCLINATIONS AND ORIFICE HEIGHTS

Effect of the Jet Stream on Pavements

Once the temperature distribution along the exhaust stream of a jet engine has been established, the maximum air-stream temperature likely to be reached at the runway surface can be estimated by drawing a line on the temperature distribution diagram, corresponding to the runway surface, and reading off the highest parameter intersected by this line. The presence of the ground will, of course, have some effect on the air flow but it is considered that, for the purpose of assessing the effect of the jet stream on runway surfacing, this can be ignored.

Fig. 7


TEMPERATURE: °C
500 450 400 350 300 275 250 225 200 175 150 125 100 90
80
70
60
50
ANGLE OF INCLINATION OF AXIS OF ENGINE TO PAVEMENT: DEGREES
0 1 2 3 4 5 6 7 8 9 10 11 12
1 2 3 4 5 6 7
HEIGHT OF JET ORIFICE ABOVE PAVEMENT IN ORIFICE DIAMETERS

Maximum Jet-Stream Temperatures at the Runway Surface for Various Engine Inclinations and Orifice Heights for $T_j = 675°$ C., $T_o = 20°$ C.

If a series of ground lines are drawn corresponding to a range of engine inclinations and orifice heights, maximum jet-stream parameters can be read off in each case and *Fig. 6* shows a convenient way of presenting this information. The parameters can readily be converted into actual temperatures once values for T_j and T_o are known; *Fig. 7* has been prepared assuming limiting values of 675° C. and 20° C. respectively for these temperatures, and from this graph the maximum temperature of the jet stream impinging on the runway can be obtained for any aircraft if its orifice diameter, height of orifice above ground level, and engine inclination are known. Conversely, if an assessment of the effect of any

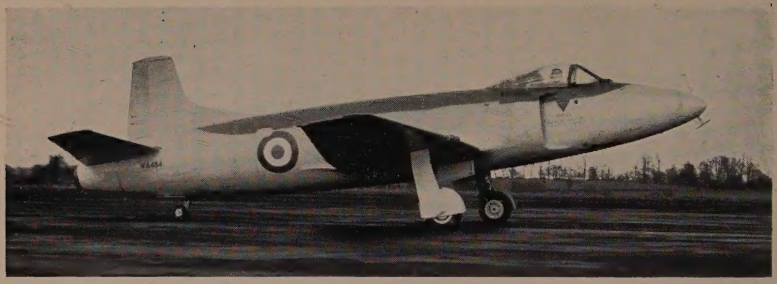
Fig. 1

EFFECT OF VAMPIRE JET STREAM ON JOINT-SEALING COMPOUNDS IN 10-FOOT-SQUARE BAYS OF CONCRETE

Fig. 2

TYPICAL SMEARS LEFT ON ASPHALT SURFACE USED BY VAMPIRE AIRCRAFT

Fig. 3 (a)



ATTACKER

Fig. 3 (b)

METEOR

Fig. 3 (c)

SABRE

Fig. 4

Stripping of Single Asphalt Wearing-Course from Concrete by Jet Aircraft on Maintenance Area

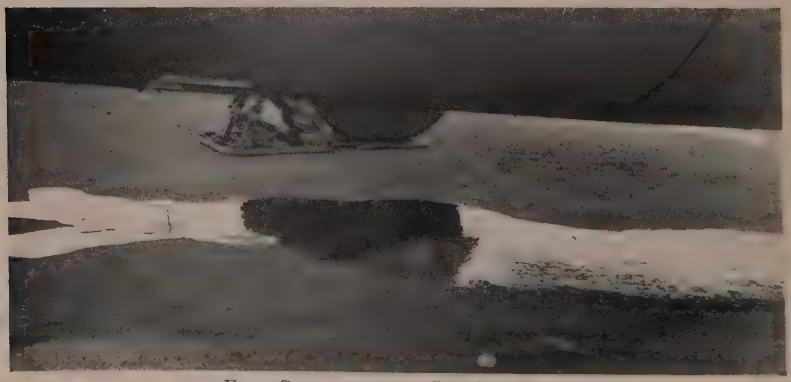
Fig. 8

Methods of Locating Thermocouples for Tests on Concrete Slabs

Fig. 15

Mobile Pouring Unit for Joint-Sealing Compounds

Fig. 16



Fuel Spillage from Jet Aircraft

aircraft on a particular pavement is required, an aircraft such as the Meteor, which would not normally cause damage, can be tilted to reproduce the required temperatures.

The engine inclination will be affected to some extent by the condition of loading of the aircraft, and in the case of tricycle undercarriages it has been noted that there is a distinct tendency for the nose of a stationary aircraft to dip a few degrees when the engine is opened up to maximum power. The tail of the Vampire, for instance, starts to lift when the engine speed reaches about three-quarters of its maximum. This reduces the inclination of the engine and the point of impingement of the jet stream on the runway moves farther away from the orifice. For this reason the effect of the jet stream on the runway appears to be much the same whether the engine is operated at full speed or three-quarter speed although it occurs at a different point in relation to the position of the aircraft.

Once an aircraft starts to move, any point in its path on the runway will be subjected to the full force of the jet stream for only a very short period and there is therefore less chance of the runway being damaged. A possible exception to this may occur in the case of aircraft, particularly those with low-slung engines, on the take-off run immediately prior to being airborne, when the nose of the aircraft is lifted, thus increasing the effective engine inclination and reducing the distance between the orifice and the pavement by an amount which may overcome the compensatory effect of the speed of the aircraft.

Tests on Rigid Pavements

In order to establish the effects of the exhaust stream on a typical concrete pavement, tests were carried out at Cardington with a Meteor aircraft (Derwent Mark I engine). Arrangements for determining these effects were as follows :—

(1) Preliminary surface-temperature measurements by means of thermetric paints.
(2) Temperature measurements at the surface of the slabs and at various depths by means of thermocouples.
(3) Visual observation of the effects of the jet stream on the slabs.

For the visual test areas twelve slabs 15 feet square, three slabs 10 feet by 5 feet, and three slabs 5 feet square were laid to a depth of 6 inches ; for the thermocouple temperature measurements a 10-foot-by-45-foot lane of 8 inches depth was laid and subdivided into lengths of 8, 10, 12, and 15 feet. The slabs were laid on a rolled base of small clinker, 2 inches thick, on a clay subsoil and the full-depth joints between the slabs were filled with an impregnated fibre-board type of filler to within 1 inch of the surface. The aggregate used for the concrete was a gravel obtained from

a pit in the Bedford area and was combined to give the following nominal grading :—

Aggregate size	$1\frac{1}{2}''-\frac{3}{4}''$	$\frac{3}{4}''-\frac{3}{8}''$	$\frac{3}{8}''-\frac{3}{16}''$	$\frac{3}{16}''$ down
Per cent	37	15	12	36

Ordinary Portland cement conforming to B.S. 12 : 1947 was used. Compressive-strength tests gave values of 4,800 lb. and 6,950 lb. per square inch at 7 and 28 days respectively. The aggregate was batched as accurately as possible by volume, the designed aggregate/cement ratio by weight being 4·8 to 1. A 7/5 tilting-drum mixer was used and sufficient water was added to produce a slump of 1 inch—corresponding to an estimated water/cement ratio of 0·45 ; 6-inch-cube compressive strengths of samples from two different batches were 5,750 and 5,150 lb. per square inch at 7 days, and 6,900 and 6,300 lb. per square inch at 28 days.

The concrete was spread and tamped by hand and the surface was sprayed with a membrane-type curing compound at the rate of 20 square yards to the gallon. Five cores were cut from the slabs and gave equivalent 28-day cube-strengths ranging between 5,750 and 6,650 lb. per square inch with an average of 6,300 lb. per square inch.

Thermetric Paint Tests

A preliminary investigation into the distribution of pavement surface-temperature in the jet stream was carried out by means of thermetric paints. These are specially prepared paints which change colour when heated to a certain temperature in 2 minutes. Various grades of paint were available with critical temperature ranging from 70° C. to 300° C. Appropriate grades of paint were applied to the surface of the slabs in 3-inch-wide strips running at right angles to the direction of the jet stream. One engine of the Meteor was directed on to the test area and was run at maximum speed for 2 minutes, the normal engine inclination being zero. The test was repeated with the tail of the Meteor pulled down so that information was obtained for engine inclinations of 4, 8, and 12 degrees. In the case of the run at zero inclination no temperatures in excess of 70° C. were registered, and for the other inclinations the paints indicated that the maximum temperatures attained were in excess of 70° C., 200° C., and 220° C. respectively. These results show reasonable agreement with values estimated from *Fig. 7*.

Thermocouple Measurements

For the thermocouple temperature measurements, eighty-four thermocouples were used and located as shown in Table 1.

The thermocouples consisted of double cotton-covered copper and

TABLE 1

Distance below surface : inches	No. of thermocouples
0	33
½	21
1	15
3	9
7	6

Eureka wires of No. 26 S.W.G. Two standard lengths of couple, 42 feet and 67 feet, were employed to reduce the amount of work involved in making up the couples and subsequent calibration. The couples selected for recording were connected to alternate contacts of a 121-way double-pole switch of low contact-resistance and negligible contact-potential. The main leads from the switch poles were fed to a cathode-ray oscilloscope. The vertical deflexion of the oscilloscope spot, a measure of the change in potential between consecutive switch positions, was recorded by a camera. The switch was rotated at a speed of ½ revolution per minute by means of a suitably geared fractional-horse-power motor; this allowed approximately one second per switch position, and the rate of travel of the film in the camera was adjusted so that each switch position gave a trace about ¼ inch in length. In order to check the recording equipment, a calibration voltage was supplied from a potentiometer to a number of positions on the switch. The hot junctions of the thermocouples were mounted on 11-inch-by-4-inch-by-4-inch precast concrete pillars. These pillars were cast with horizontal chases at ½, 1, 3, and 7 inches from the top, so that the first 4 inches of the thermocouple leads were kept at the same level and temperature. The pillars were set out on a 3-inch concrete base-slab, and the leads from each pillar were run in rubber tubing to the main switch (see *Fig. 8*). Another 3-inch layer of high-slump concrete was then poured to protect the leads and to fix the blocks rigidly before the main 8-inch slab was placed.

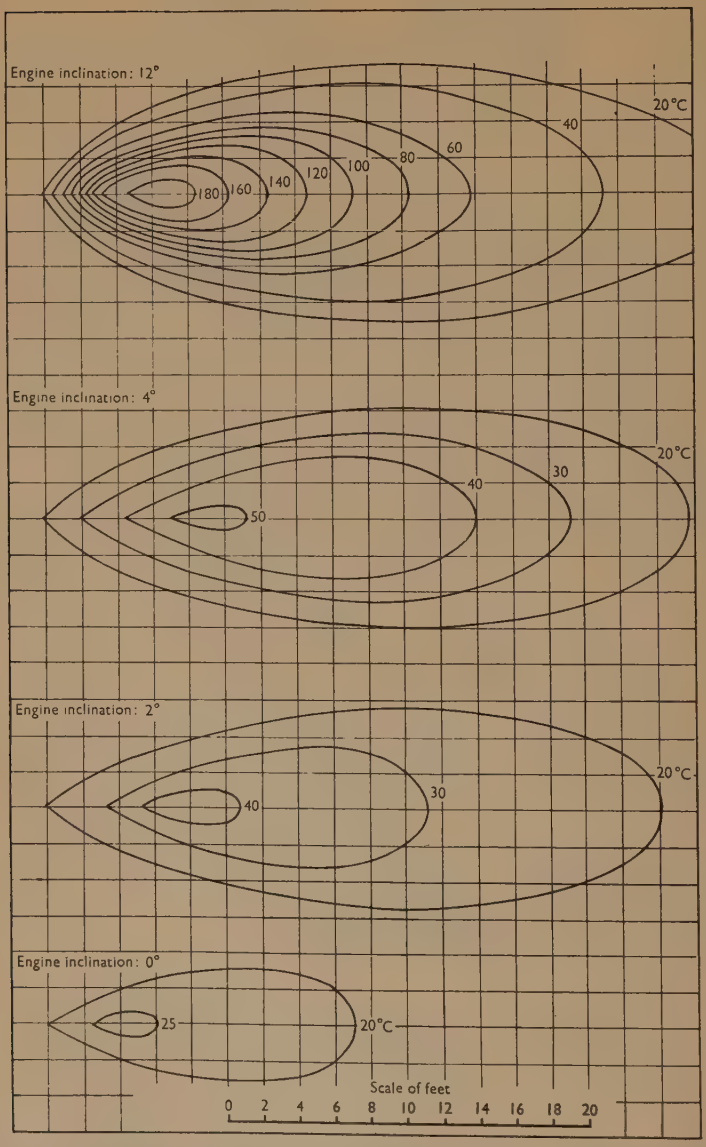
Figs 9 and *10* show typical results obtained for the horizontal and vertical temperature distribution in the slab.

Visual Tests

For the visual tests, which took place about 2 months after the concrete had been laid, bays of various sizes were subjected to the Meteor jet stream with the engine inclined at 0, 4, 8, and 12 degrees. These bays were surrounded by full-depth joints. The jet stream was, unless otherwise stated, directed along the centre-line of the slab, and the aircraft was so positioned that the area of maximum temperature occurred roughly in the centre of the slab. The effects noted are summarized in Table 2.

Cores cut from the line of the cracks showed that they extended the full depth of the slab; the cracks also appeared to be more clearly defined

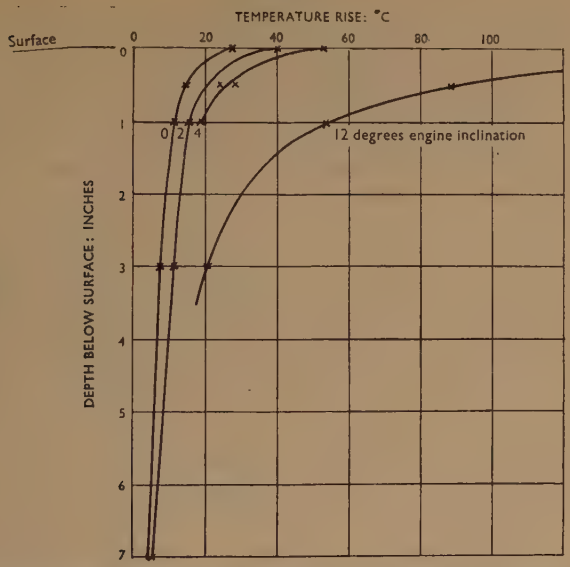
Fig. 9



TEMPERATURE CONTOURS OBTAINED FROM SURFACE THERMOCOUPLE MEASUREMENTS IN A CONCRETE SLAB AFTER RUNNING A METEOR AT FULL OUTPUT FOR 6 MINUTES AT VARIOUS ENGINE INCLINATIONS

(All temperatures shown are rises above ambient)

Fig. 10



MAXIMUM RECORDED TEMPERATURE RISES IN AN 8-INCH CONCRETE SLAB AFTER RUNNING A METEOR AT FULL OUTPUT FOR 6 MINUTES AT VARIOUS ENGINE INCLINATIONS

at the bottom than at the top of the slab. It was also found that the surface crazing observed in the prolonged tests extended to a depth of less than $\frac{1}{4}$ inch.

It would appear, therefore, that damage to concrete pavements can take three different forms :—

(1) Full-depth cracking, presumably arising from thermal stresses in the slab as a whole. The extent of this cracking would depend upon the surface temperature attained, the dimensions of the slab, and the physical properties of the concrete.

(2) Surface crazing of the mortar. This is probably caused by the excessively rapid drying out of the surface layer of mortar when exposed to the jet stream at a temperature of 250° C. to 300° C. for relatively long periods.

(3) Surface spalling. This is the only type of damage to high-grade concrete which has so far been reported from a site, and the information available suggests that the surface of the concrete had been subjected to a temperature of about 350° C.

TESTS ON FLEXIBLE PAVEMENTS

Bays of stone-filled asphalt, sand asphalt, fine tarmacadam, and coarse tarmacadam were also laid at Cardington for trials with a Meteor aircraft.

TABLE 2

Bay size : ft × ft	Inclination of engine : degrees	Maximum temperature of impinging gases : °C.	Time of running at maximum output : minutes	Observed effect
15 × 15	0	55	8	No damage
15 × 15	4	90	8	No damage
15 × 15	8	170	Four periods of 1 min. at 3-hour intervals	Single crack appeared at the edge of the slab farthest from engine, running towards the centre of the slab
15 × 15	12	280	1	Single crack running from the slab edge towards the centre
15 × 15	12	280	121 in periods ranging from 1 to 10	No spalling. Several main cracks produced including one in an adjacent bay. Some surface crazing developed after about 1 hour total running time
15 × 15	12	280	8	Jet stream directed along joint between two bays. No damage
10 × 5	8	170	5	No damage
15 × 5	12	280	2	Single crack running from the slab edge towards the centre
5 × 5	12	280	3	Single crack running from the slab edge towards the centre
10 × 5 and 5 × 5	12	280	Less than 2	No damage

The trials again consisted of both visual observations and thermocouple temperature measurements for a range of inclinations. Generally the engine was run at maximum speed for periods of 6 minutes, but in some cases a number of $\frac{1}{2}$-minute runs were done in quick succession to determine the accumulative effect of repeated applications. For the temperature measurements, thermocouples were located both at the surface and at the base of the carpet under test, and were suitably patterned to record temperature over the whole area of impingement of the exhaust gases. The method of measurement was as previously described for the

concrete slabs. The general bulk softening of the carpets was assessed by prodding with a sharp point as soon as possible after the engine was turned off. To assist in the assessment of the degree of softening, a simple penetrometer, consisting of a loaded wooden spike carried on two long rods, was also used. This device operated while the engine was running, and the jet stream vibrated the spike sufficiently for it to make an impression on the carpet. The depth of the impression proved to be a good assessment of the degree of surface softening. The results obtained were as follows :—

Stone-Filled Asphalt

The pavement consisted of a $\frac{3}{4}$-inch consolidated thickness of stone-filled asphalt laid on a base course of $2\frac{1}{2}$ inches of coarse-graded tar-macadam. For the asphalt the maximum size of the coarse aggregate was $\frac{1}{2}$ inch and the mix proportions by weight were 43 per cent coarse aggregate, 50 per cent fine aggregate, and 7 per cent filler, mixed with 180/220 penetration straight-run bitumen at the rate of 14–16 gallons per ton of dry aggregate including filler. A 2–3-ton roller was used for initial rolling and an 8-ton roller for final rolling. *Fig. 11* shows the effect of 6-minute runs on the temperature at the surface and at a depth of $\frac{3}{4}$ inch for engine inclinations of 0, 2, and 4 degrees. Examination of the carpet immediately after each of the 6-minute runs showed the effects listed in Table 3.

TABLE 3

Engine inclination : degrees	Condition of pavement immediately after run	Ambient temperature : ° C.
0	No appreciable softening	16
2	No appreciable softening	18
4	Softening from 15 to 38 ft	7
6	Softening from 12 to 42 ft from the jet pipe	8

Tests using the penetrometer to assess the degree of softening gave the results shown in Table 4, the ambient temperature at the time being 12° C.

A further series of tests was then undertaken to determine the effect of short runs. For these tests the engine speed was increased from idling to full over a period of 30 seconds, and maintained at full output for 15 seconds ; the ambient temperature was 12° C. The observations are shown in Table 5.

Fig. 12 shows the temperature induced by five $\frac{1}{2}$-minute runs in quick succession for various engine inclinations.

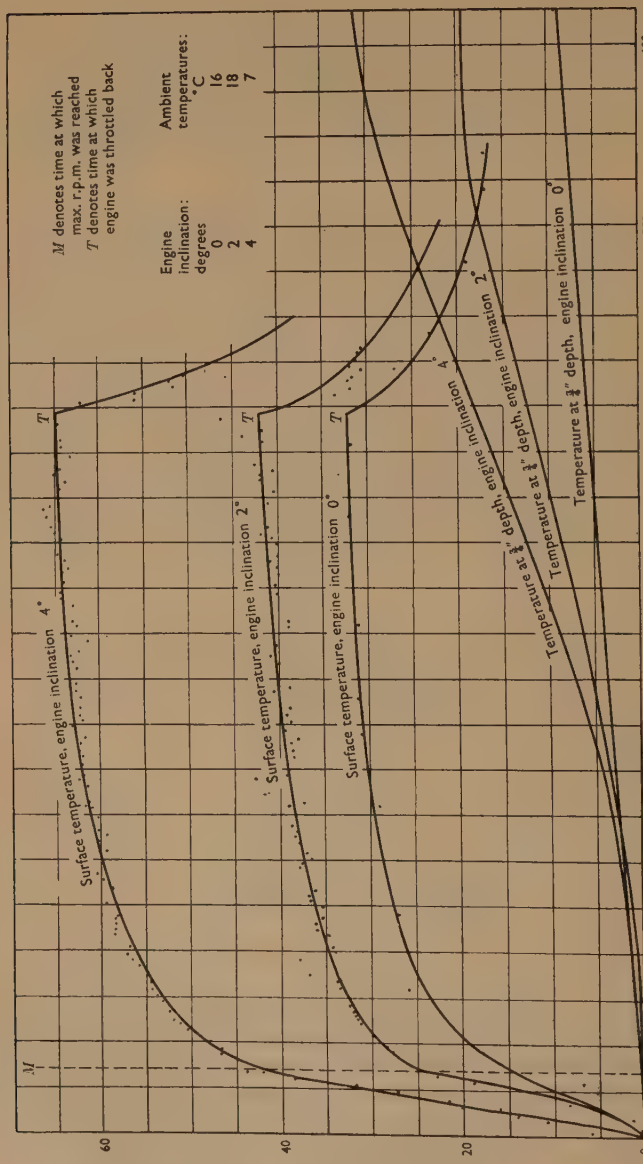


Fig. 11

TEMPERATURE RECORDED IN A ¾-INCH STONE-FILLED ASPHALT CARPET. 6-MINUTE RUNS AT FULL OUTPUT WITH METEOR AT VARIOUS ENGINE INCLINATIONS

TABLE 4

Engine inclination : degrees	Observed effect
2	No appreciable marking after 6 minutes
3	Appreciable marking occurred after 2 mins 45 secs
4	Surface marked to a depth of $\frac{1}{4}''$ and some material blown away on being disturbed after 1 min. 15 secs

TABLE 5

Engine inclination : degrees	Observed effect
6	Very slight softening up to a depth of $\frac{1}{8}''$. No sign of surface smearing. Softening rapidly disappeared when the engine was switched off
7	Bitumen on the surface very slightly smeared
8	Bitumen on the surface smeared and carbonized

The foregoing tests showed that the limiting conditions for long and short running periods were somewhat different. In the case of long running periods, the bulk stability of the carpet would be seriously affected by softening caused by the rise in temperature throughout the carpet, and aircraft passing over areas so affected would probably cause rutting. For short running periods, however, a considerably higher jet-stream temperature would be required to cause bulk softening, since the pavement-surface temperature is likely to reach only about three-quarters of the jet-stream temperature and this temperature rise would probably not extend to any appreciable depth. Limiting conditions for short running, therefore, are reached when the binder on the surface starts to smear, since repeated application of this treatment would result in progressive deterioration of the wearing course.

In the case of stone-filled asphalt, the Meteor tests showed that these limiting conditions were reached at engine inclinations of $2\frac{1}{2}$ degrees and $6\frac{1}{2}$ degrees for long and short running periods respectively.

Hot-Mix Sand Asphalt

The carpet consisted of a $\frac{1}{4}$-inch-down sand containing about 10 per cent of filler mixed with 180/220 penetration straight-run bitumen at the rate of 16–18 gallons per ton of dry aggregate. Unfortunately the thermocouples in this carpet were disturbed during the laying and the temperatures recorded were unreliable. However, numerous observations

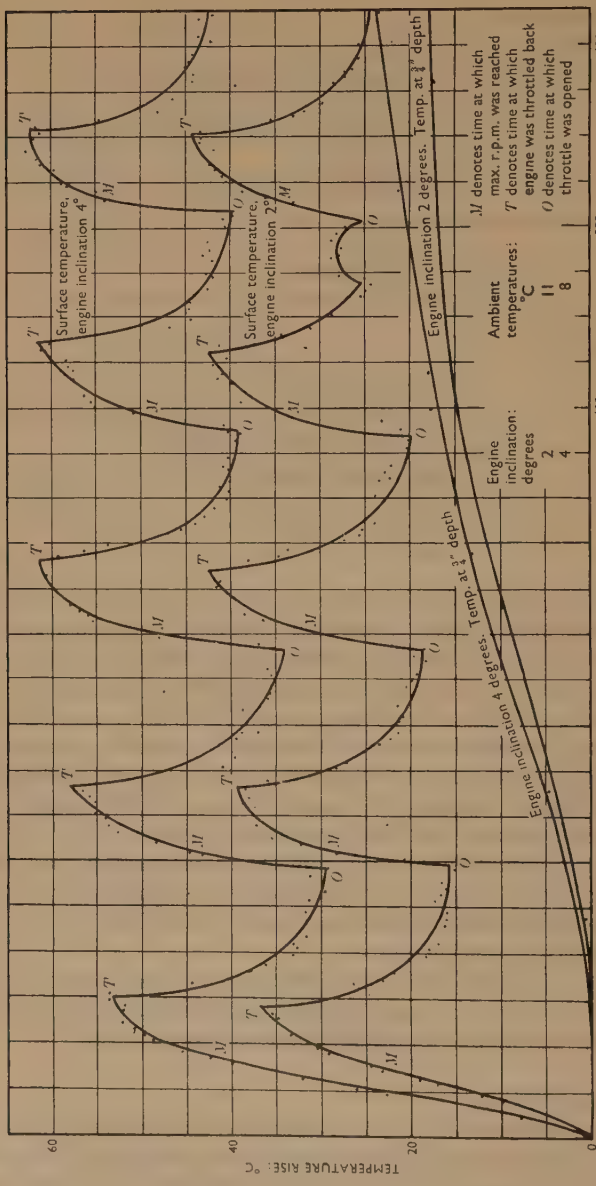
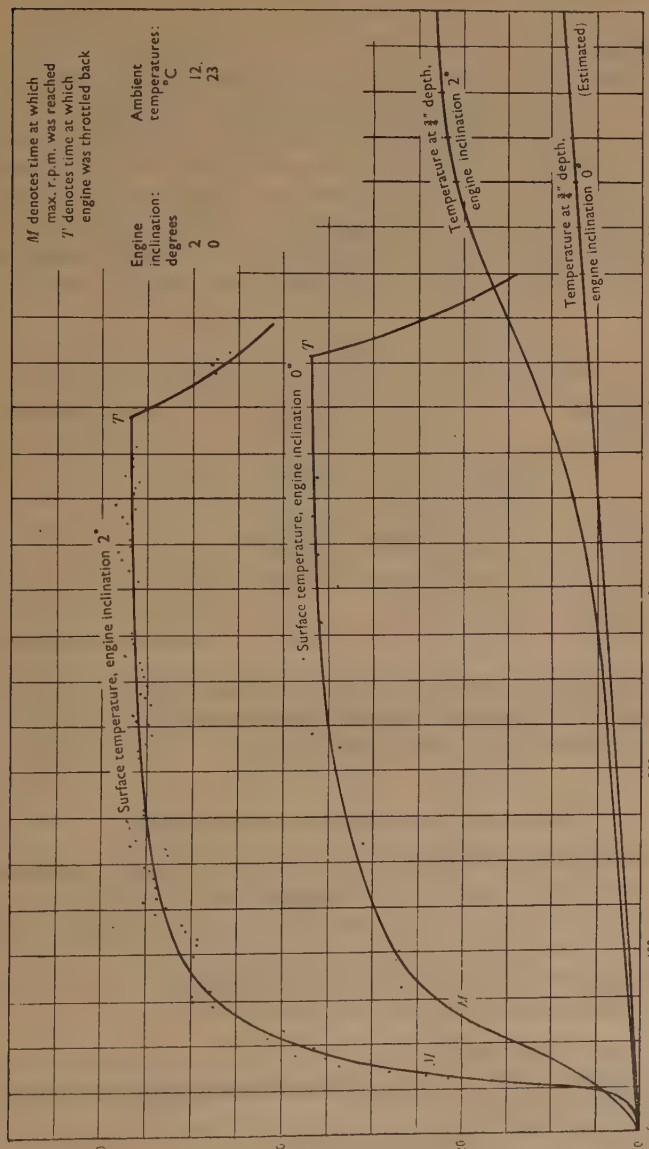


Fig. 12

TEMPERATURES RECORDED IN A ¾-INCH STONE-FILLED ASPHALT CARPET. 5-MINUTE RUNS AT FULL OUTPUT WITH METEOR AT VARIOUS ENGINE INCLINATIONS

Fig. 13



TEMPERATURES RECORDED IN A ¾-INCH FINE TARMACADAM CARPET. 6-MINUTE RUNS AT FULL OUTPUT WITH METEOR AT VARIOUS ENGINE INCLINATIONS

of the pavement indicated that its performance was little different from that of the stone-filled asphalt.

Fine Tarmacadam

The surfacing was laid to a depth of $\frac{3}{4}$ inch on a $2\frac{1}{2}$-inch base course of tarmacadam. The maximum size of the aggregate was $\frac{1}{4}$ inch; the binder was a type " B " tar (to B.S. 76 : 43) with a viscosity of 140–180 seconds at 30° C., and was mixed with the aggregate at the rate of 12 gallons per ton of dry aggregate. *Fig. 13* shows the rate at which temperature builds up in the surface and base of the carpet for 6-minute runs at engine inclinations of 0 and 2 degrees. Examination of the carpet immediately after each of the 6-minute runs showed the effects listed in Table 6.

TABLE 6

Engine inclination : degrees	Condition of pavement immediately after run	Ambient temperature : ° C.
0	No appreciable softening	23
2	Appreciable softening from 18 to 34 ft from the jet pipe	17
4	Surface stripped after 1 minute	17

Tests to determine the limiting engine inclinations as previously described in the case of the stone-filled asphalt carpet gave values of $1\frac{1}{2}$ and $3\frac{1}{2}$ degrees for long and short runs respectively.

PRACTICAL APPLICATION OF FIELD TESTS

The tests carried out indicated that, if a suitable joint-sealing compound can be provided, high-grade concrete appears to be a satisfactory form of construction for all types of jet aircraft at present operating in Great Britain.

Existing types of bituminous pavements are likely to suffer different degrees of damage depending on the following factors :

(1) engine inclination,
(2) height of orifice above pavement in terms of orifice diameters,
(3) orifice temperature,
(4) duration of engine-running period,
and (5) intensity of traffic.

The likely periods of stationary running of the engines under operational conditions fall into the following two categories :—

(*a*) Long running periods, which occur on standing and maintenance areas, generally for servicing checks, and may amount to as much as 6 minutes running at full output.

(*b*) Short running periods, which occur at the ends of runways and adjacent taxi-tracks. It is difficult to establish a definite period here, because conditions may be very variable. For instance, an experienced pilot would need less time to position and move off than, say, a pupil on a training or instrument take-off. Again, in mass formation take-offs, which are quite common with military aircraft, the aircraft at the rear of the formation may of necessity be standing at full output for longer periods than the formation leader. Experience resulting from observation and discussions with flying personnel indicate that the short running periods at full output will normally be of about 15 seconds duration, although on rare occasions may be as long as 1 minute.

Lines representing critical jet-stream temperatures for long and short running periods can be drawn on *Fig. 7* by referring to the critical Meteor engine-inclinations already established for stone-filled asphalt and tarmacadam. As previously explained, in the case of short running periods the surface of the pavement will not have attained the temperature of the impinging gases. *Fig. 14* shows the diagram so obtained; the probable effect of any jet aircraft on the pavements considered can be assessed from this diagram by plotting the point corresponding to the engine inclination and height of orifice above the ground. For example, the point corresponding to the Vampire with a Goblin Mark 2 engine would fall just above the line representing limiting conditions for short running periods on asphalt surfacing. The Vampire would therefore be expected to damage both asphalt and tarmacadam subjected to short running periods at full output. This assessment is borne out in practice.

If all types of jet-propelled aircraft are to operate without restriction on the existing types of bituminous surfacings some form of protection against the destructive effect of the jet efflux will be necessary, particularly at runway ends and servicing areas. Such a protective carpet would, of course, have to resist the solvent action of the fuels.

Development of Jet-Fuel-Resistant Joint-Sealing Compounds

The main requirement of a joint-sealing compound hitherto has been to provide a flexible seal in concrete joints which will not soften unduly in hot weather but will continue to cohere at low temperatures and prevent the entry of water and grit into the joint.

As already explained, the rubber-bitumen compounds which were in general use in concrete pavements offer little resistance to the solvent

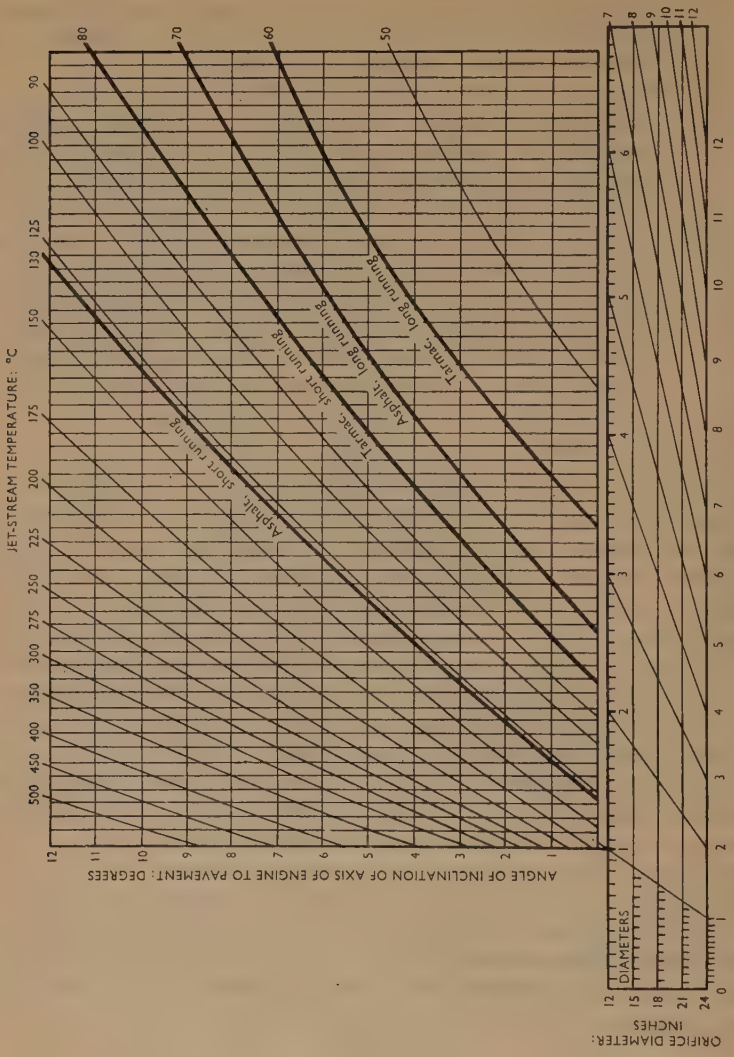

JET-STREAM TEMPERATURE: °C
ANGLE OF INCLINATION OF AXIS OF ENGINE TO PAVEMENT: DEGREES
HEIGHT OF JET ORIFICE ABOVE PAVEMENT: FEET
ORIFICE DIAMETER: INCHES
DIAMETERS
Tarmac, long running
Asphalt, long running
Tarmac, short running
Asphalt, short running

Fig. 14

NOMOGRAM FOR ESTIMATION OF PROBABLE EFFECT OF ANY JET AIRCRAFT ON TARMACADAM AND ASPHALT

action of jet fuels. The first essential, therefore, was to produce a joint-sealing compound with properties similar to those of the rubber-bitumens but which would be resistant to these fuels. With this in mind, products based on selected grades of coal-tar and pitch were developed and showed considerable promise when tested in the laboratory.

In order to assess the suitability of any joint-sealing compound, laboratory tests based on those previously described by Millard and

Critchell [3] were carried out to give an indication of its low-temperature ductility and adhesion, resistance to ingress of grit, and resistance to the solvent action of fuel. If the results of the laboratory tests proved satisfactory, small-scale site trials were arranged so that the performance of the material under service conditions could be observed.

Resistance to the solvent action of the fuel is indicated by measuring the loss in weight and increase in penetration of the compound after about 20 cubic centimetres of fuel have been standing on top of the compound in a standard penetration dish at room temperature in the laboratory for 7 days. A lid is placed on top of the dish for this period to prevent excessive evaporation of the volatiles in the fuels. For the laboratory tests a specially prepared sample of fuel is used, which contains the maximum aromatic content permitted by the Ministry of Supply fuel specification.

Approximately 20-per-cent loss in weight together with a 100-per-cent increase in the penetration at 25° C. is common to the normal rubber-bitumen joint-sealing compounds when tested under such conditions.

It has been possible to produce an apparently satisfactory fuel-resistant joint-sealing compound. Site trials have shown, however, that, when subjected to the jet efflux of the Vampire, compounds having a " vertical flow test " value as low as 5 per cent start to flow when the engine is idling and are displaced in varying degrees at maximum output. It was noted, however, that with these compounds it was possible to walk on the displaced material 30 to 45 seconds after the test without any signs of tackiness. If the occasional displacement of the fuel-resisting joint-sealing compounds by the more severe types of jet aircraft, together with the additional maintenance costs, is acceptable, then these types of compounds will prove satisfactory. Should this not be acceptable, however, it is felt that there will be no alternative other than to provide the compound with some form of protection from the jet efflux, particularly at the runway ends and on servicing areas. Various materials have been investigated for this purpose, the two most successful being impregnated rope, and a cork-cement composition. The former satisfactorily withstood two 1-minute blasts followed by a 2-minute blast from a Vampire engine at full output. However, much more investigation is still required in this field.

The new fuel-resisting compounds are more expensive than the rubber-bitumens, and also more difficult to pour. In order to ensure that the material is not over-heated and is poured at the correct temperature, mobile pouring units such as the type shown in *Fig. 15* should be used. The advantage of such a unit is that the material can be poured directly into the joint without being spilt on the adjacent slab surfaces, thereby reducing the area of sealing compound exposed to the jet efflux.

Protective Surfacings

The temperature measurements taken have shown that a $\frac{3}{4}$-inch-thick surfacing, with normal aggregates provides sufficient thermal insulation to

prevent the softening of asphalt by heat from the jet efflux of any type of aircraft at present in service, so that special heat-insulating materials need not necessarily be considered for protective surfacings. A satisfactory protective surfacing should in addition have the following properties :—

(1) Resistance to the combined effect of the heat and blast from the jet efflux.
(2) Resistance to the solvent action of the fuel.
(3) Sufficient flexibility to follow the movement of a flexible pavement under heavy moving loads without cracking.
(4) Freedom from shrinkage cracking.
(5) Adhesion to the existing surface.
(6) A durable wearing surface.
(7) Ease of mixing and emplacement by existing plant.

In attempts to meet these requirements, various types of protective carpets were laid on asphalt surfaces at Cardington. They were tested with a Meteor aircraft at engine inclinations ranging, in intervals of 2 degrees, from zero to 12 degrees—corresponding to jet-stream temperatures of 55, 65, 90, 130, 180, 230, and 290° C. The following results were obtained.

Bituminous carpets.—Two types of bituminous carpet were tested. One was a mastic asphalt and the other a cold-laid mix using limestone fine-aggregate Portland cement and bituminous emulsion. Both of these carpets were intended primarily to resist the heat and blast of the jet stream. Tests with the Meteor for 6 minutes on the mastic asphalt showed no effect at engine inclinations of 2 and 4 degrees, but at 6 and 8 degrees the surface smeared and ripples were formed which, in the case of the 8-degree test, were up to $\frac{1}{4}$ inch deep. Similar tests on the emulsion-based carpet showed that at 6 degrees engine inclination the surface softened and became rather friable ; at 9 degrees, similar effects were noted, but to a somewhat greater degree. Although the behaviour of these two carpets under the jet stream is slightly better than the normal asphalt, they are both very susceptible to the solvent action of the fuels.

Asbestos cement.—An area of asbestos-cement screed was laid by hand to a thickness of $\frac{1}{2}$ inch. The mortar, which contained about 50 per cent of asbestos, was laid on a bituminous emulsion tack-coat. Tests with the Meteor took place about 4 weeks after laying ; 6-minute runs with the aircraft's engines inclined at 0, 2, and 4 degrees caused no damage. A subsequent run at 12 degrees loosened a small area after about 3 minutes, and after 6 minutes the carpet cracked and a piece about 6 feet by 3 feet was blown away. There was found to be little adhesion between the asbestos cement and the asphalt. Because of the fibrous nature of the material, its flexibility under load would probably be adequate, but for

the same reason it is not likely to provide a good wearing surface and, being an absorbent material, would be susceptible to frost damage.

Cement and impregnated wooden chips.—This carpet consisted of a mortar of cement and ½- to 1-inch impregnated wooden chips, and was laid by hand in very cold weather to a depth of 1 inch. It was covered with asbestos-cement sheeting for 4 days after laying, to give some protection from the weather. Although no cracking could be seen immediately after the carpet had set, some form of curing would obviously be necessary in warm dry weather. After 3 weeks, Meteor tests of 6 minutes duration at engine inclinations of 6, 8, and 10 degrees caused no damage to the carpets. After a 6-minute run at 12 degrees inclination, however, map cracking to a depth of ¼ to ½ inch was observed on the surface. Adhesion to the asphalt base appeared adequate, but it was subsequently found that excessive drying shrinkage caused all four corners of the slab to curl up, damaging the existing asphalt which adhered to the under side of it.

Plastics and resinous materials.—Various thicknesses of polyvinyl chloride sheeting were tried, but the difficulty in this case was the tendency of the material to lift at the joints under the action of the blast. Fuel spillage, although not likely to affect the sheeting itself, would make the smooth surface dangerously slippery.

Vermiculite.—A number of bays of a cement-vermiculite mortar screed were laid to thicknesses of ½ inch and ¾ inch. Ordinary Portland cement was used for half of the bays and high-alumina cement for the remainder; in each case the material was sprayed on to the asphalt and there was a further sub-division between the bays left as sprayed and those consolidated to a smooth surface by means of a beam vibrator. A resinous-membrane type of curing compound was used throughout. Tests with the Meteor showed that with engine inclinations of up to 9 degrees, no cracking took place, though some spalling was noticed on the vibrated ½-inch bay containing ordinary Portland cement. At 12 degrees inclination, cracking occurred in the bays containing high-alumina cement, but with the exception of the bay which had previously spalled the ordinary Portland-cement bays appeared to be unaffected. As with other materials which are primarily heat insulators, the porous nature of the aggregate leads to poor resistance to frost action, and in fact the surfaces of all the bays became very friable during the first winter after laying.

Protective paints.—Various types of paints have been produced to protect asphalt surfaces from the solvent action of the fuel. It seems very unlikely, however, that any paint film will be sufficiently durable to provide a complete seal on an asphalt surface taking heavy traffic. The fuel will readily penetrate any minute break in the film to soften the asphalt and render the remainder of the treatment useless.

Tar carpets.—Two carpets based on coal-tar or pitch have been tested. The first type was a cold-laid carpet, the binder being a mixture of tar

and pitch emulsions. A crushed limestone aggregate graded $\frac{1}{8}$ inch to dust was used, and " 417 "-type cement was added. Various mix proportions were tried, and the most satisfactory bay showed no damage when tested with the Meteor for 2 minutes at an engine inclination of 4 degrees. There was no appreciable softening at 6 degrees, and at 9 degrees there was some slight pitting noted, leaving small holes about $\frac{1}{2}$ inch deep in the surface. Although not completely immune, this carpet was found to be reasonably resistant to AVTAG, and it was therefore decided to lay trial areas on an airfield; " 417 "-type cement was again used for these trials. Subsequent inspection showed that shrinkage cracking had occurred, resulting in the carpet being reduced to approximately 10–15-foot bays. In an effort to eliminate this cracking, the mix proportions were modified and a continuous lane, 300 feet long, was laid to a width of 12 feet. Here again it was found that regular transverse cracking occurred at 10–15-foot centres. Investigations are now proceeding with a view to either eliminating or controlling this cracking.

The second type of carpet laid was a hot mix to the normal Air Ministry asphalt wearing-course specification, the binder being a modified resin-pitch combination. A limestone aggregate was used and a small amount of asbestos fibre was introduced into the mix. This carpet was very slightly softened by a one-minute test with the Meteor at 6 degrees; a 2-minute run at the same engine inclination resulted only in slight smearing and softening of the surface. Although this carpet is less resistant to the temperature of the jet efflux, it is more flexible than the cold-laid carpet, and the possibility of cracking is therefore greatly reduced. Furthermore, the introduction of the resins offers greater resistance to the solvent action of the fuels. Arrangements are now being made for an area of this material to be subjected to site trials.

Additional Problems

In addition to the problems already discussed there are a number of other problems which will require investigation.

Drainage Problems

It has already been explained that the fuels used by jet-propelled aircraft may lie about on servicing and fuelling areas for considerable periods of time. Spillage arises when the aircraft are being fuelled, either from overfilling the tanks or from faulty hose connexions, and from wet starts; furthermore when the engine is shut down, there may be as much as a pint of unused fuel remaining in the system, which will drain out on to the pavement. *Fig. 16* shows a typical example of spillage occurring immediately after an aircraft has shut down its engine. The amount of spillage naturally increases with the number of aircraft movements and may attain an embarrassing volume if strict fuelling discipline is not

maintained. It is difficult to estimate the amount of spillage which will occur on civil airfields with the introduction of large numbers of jet-propelled airliners. It has been suggested that underwing pressure refuelling will eliminate most of the spillage, but this of course remains to be seen. Fuel spillage is small over the main area of the runway pavement although rather more spillage may occur at the ends of the runways and on taxi-tracks.

A serious danger arises, however, during heavy downpours of rain when any fuel lying about the servicing areas will be carried into the stormwater drainage system. Kerosene is accumulative and cannot be disposed of through soakaways, because it would lie on the top of the water-table and possibly kill crops. Evidence of the accumulative effects of fuels has been noticed in two instances where bituminous waterproofing skins on the outside of underground buildings have been attacked. The method of dealing adequately with these problems may well prove to be of sufficient magnitude to form the subject of a separate Paper; some special precautions will certainly have to be taken.

On one recently constructed airfield, where the runway and taxi-tracks crossed a local catchment area, rather elaborate precautions were taken to ensure that no fuel penetrated through to the water-table. A series of small traps were provided in the main stormwater drainage system to separate any fuel carried off the pavement surface by stormwater before it was discharged back into the ground through soakaways. Each trap was provided with a by-pass so that in an emergency the stormwater could be directed back into the main drainage system, which entered a large trap before being discharged into a deep ditch.

Curing of Concrete

Bituminous emulsions, as described by Graham and Martin [4] for curing pavement-quality concrete on airfields, are readily attacked by jet fuels, and the resulting tacky surface would soon render such areas unusable. This has led to the increased use of resin-based curing membranes. With these membranes it is necessary to ensure a complete and uniform coverage of the newly laid concrete surface and to achieve this a very fine atomization of the liquid is required. As with bituminous emulsions, when using resin-based membranes adequate provision should be made for protecting the concrete from the harmful effects of sunshine and other elements during the early stages of hardening.

Erosion

The heat and blast from the jet engines will eventually result in grassed areas being burned up, which will raise subsequent questions such as dust control, possibly necessitating the introduction of suitable blast deflectors at the edges of standings. Terminal aprons and standings will be particularly affected with constant use. The outboard positioning of jet

engines on larger types of aircraft will also give rise to erosion problems on the grassed verges adjacent to the taxi-tracks, and some alternative form of construction may have to be provided for these areas.

Airfield Lighting

Blister (or flush)-type lighting fittings which are installed in positions where aircraft operate, are normally well able to withstand blast, but in certain circumstances heat-resisting glass may be necessary. The main trouble, however, is that with the higher tire pressures and landing speeds of jet aircraft, the strength of these fittings and their seatings has had to be increased and it is necessary to reduce their overall height and volume above the pavement surface to lessen the jolt which they cause to aircraft running over them. This problem is most acute in the case of fittings for centre-line lighting on runways.

Underground equipment, including cables, transformers, and leads to the lighting fittings, whilst not subject to blast, heat, or heavier mechanical loads, have now to be impervious to jet fuels, which as a result of spillage can contaminate the ground or ducts in which the cable is laid. Whereas ordinary tough rubber-sheathed cables gave quite satisfactory service with piston-engined aircraft, it is now necessary to use lead or plastic sheathed cables and all the gaskets and cable entries on the transformers must be of suitable materials to withstand the damaging effects of these fuels.

Conclusion

The Authors have confined their attention to the immediate problems arising and have endeavoured to produce a rational basis from which the airfield engineer can approach these new problems. It is realized that such an approach may have many shortcomings and that future developments of jet aircraft will no doubt produce even further problems. The introduction of after-burning and rocket-assisted take-offs may well result in considerably higher surface temperatures than have hitherto been experienced. Much can be done by the aircraft designer to lessen temperature effects by keeping the jet orifice as high as possible and positioning the engine axis as near the horizontal as possible. For example, a very large jet engine having a 20-inch-diameter orifice mounted with its axis horizontal and 4 feet above the pavement results in a pavement temperature (in a temperate climate) of only about 60° C. Even with after-burning, the pavement temperature induced by such an engine should not be much greater than 120° C. In the same way, this engine mounting will reduce the damaging force of the jet stream.

Operators can help considerably by ensuring that jet-propelled aircraft do not run their engines at full output for excessive periods when the aircraft are standing on areas not specifically provided for that purpose. In addition, unnecessary fuel spillage can be avoided by insistence upon a

high standard of maintenance on hoses and connexions of fuelling tankers and care taken in fuelling and servicing.

Whilst originally it was considered that, to resist the effects of jet aircraft, high-quality concrete would have to be provided on runway ends and servicing areas, it now seems possible that a suitable type of flexible construction using a modified binder may prove satisfactory. Where damage occurs on airfield pavements, additional maintenance will be incurred, and the airfield engineer will have to decide whether to accept the recurrence of this damage, or to take advantage of the work done in the development of the new materials.

Acknowledgements

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Vickers-Armstrong Limited,
Gloster Aircraft Company Limited,

for supplying, and giving permission to reproduce, the photographs of aircraft in *Figs 3* (*a*) and (*b*) also to the Royal Canadian Air Force for supplying the photograph in *Fig. 3* (*c*).

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The Paper is accompanied by nine photographs and nine sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Authors introduced the Paper with aid of a film illustrating the effects of a Vampire jet, operating at different outputs, on various kinds of surfacing.

Mr K. C. Mann said that only co-operation between aircraft designers and airfield designers would prevent the problem which had arisen through the adoption by the aircraft industry of jet engines from becoming almost insoluble. If the aircraft designers were to accentuate still further the trend that was shown in *Fig. 3* (*a*) (Attacker) by designing aircraft with even greater downward inclination of the jet, he thought that nothing short of some expensive refractory material would be necessary on the runways ; and in point of fact the jet people would put themselves out of business, because it would be too expensive to put aircraft on the ground !

The film had shown that the Vampire's jet had devastating effects. The Meteor's jet was almost horizontal, and in *Fig. 3* (*c*) it seemed that the outlet orifice of the Sabre's jet was directed upwards ; he doubted whether further development of that trend would commend itself to aircraft designers.

Although there had been a degree of co-operation between the aircraft designers and the airport designers, there remained a jet effect problem, and the film which the Authors had shown had given a very clear picture of the devastating results of a very short application of jet blast.

The curves for zero engine inclination in *Figs 9* and *10* showed that the temperature rise during a 6-minute blast from a jet engine was 25° C. on the surface of the concrete, but only 12° C. at a depth of $\frac{3}{4}$ inch below the surface ; thus there was a temperature difference between the surface and $\frac{3}{4}$ inch into the concrete of 13° C. In *Fig. 11*, it would be seen that for a stone-filled asphalt carpet under the same conditions of application of heat at zero inclination of the jet there was a 33° C. rise of surface temperature on the asphalt ; but from the lower curve it would be seen that $\frac{3}{4}$ inch underneath the surface—that was to say, at the under-surface of the asphalt—the temperature rise was 6° C. ; so that there was a temperature difference of 27° C. between the surface and the underside of the asphalt. Again, in *Fig. 13*, where the curve referred to a fine tarmacadam carpet, the temperature rise at the surface was higher still, at 37° C., and again on the underside of the carpet, $\frac{3}{4}$ inch thick, it was only 6° C. ; so that there was a temperature difference of 31° C. between the surface and the underside of the carpet.

It seemed to Mr Mann that the concrete in *Fig. 11* (supposing that it had been reasonably smooth and white), gave 8° C. less temperature rise on the surface than the asphalt and 12° C. less than tarmacadam ; it was, however, apparently much more conductive of heat. He thought, therefore, that the black top should be smooth to reduce the heat absorption

nd softening. It appeared from the figures in the Paper to be a much etter insulator, thickness for thickness, than concrete, and those figures dded strength to the Air Ministry's understanding of rigid (concrete) avement design, namely, that the elimination of temperature stresses (for hich purpose they had adopted black top) would give a more economic esign. The jet tests had more or less shown that the Air Ministry was ght in believing that there was a considerable insulating property in lack top, and that it was more efficient if it had a smooth surface.

The Authors had also given an account of the efforts made to find heat- nd fuel-proof surfacings and jointings. He could add to that by saying nat, to his knowledge, forty different classes of materials had been xamined in detail by the Air Ministry. Many suggestions had been ade: for instance, it had been suggested that a certain by-product of aper pulping would be efficacious, but it had been found that if it got et it just went into a pulp or jelly. Another proposal had been that a aterial known as dead-burnt magnesite would be very good because it as a refractory material, but investigations had shown that it was an xcellent conductor of heat within the limits of the temperatures involved, that the material was not of much use. There had also been the ifficulty of how to bind the magnesite on the surface of the pavement and keep it together. It had been suggested that animal blood was a very ood medium for binding, but that suggestion had not been pursued any rther.

Mr Mann hoped that a solution had been found to the problem of ealing with the jet effects from aircraft in use and those which were xpected to come into service in the next few years, but it was still neces- ry to look ahead. Hoverplanes or helicopters were becoming more and ore used for passenger traffic and, in war, for getting into isolated and fficult places, but they were still cumbersome-looking machines, and he elieved that in the course of time the designers of aircraft engines would erfect what was termed the "vertical jet." What was going to be the fect of such aircraft on the pavements? Would the aircraft take off nd climb vertically, or would they go up, as he expected, at a fairly steep ngle? If a jet was directed vertically on to the airfield surface, there ould be a temperature of the order of 600° C. underneath the aircraft— hich certainly none of the materials so far examined would withstand. would mean the use of some furnace refractory material.

Mr Mann pointed out that fuel spillage, in the case of vertical-jet air- aft, would probably be disastrous; but perhaps the development of omic propulsion would obviate that difficulty!

Mr P. L. Critchell observed that the temperature contour diagrams *ig. 9*) would serve as a useful indicator to the aircraft designer of the es along which it was undesirable for him to proceed in the interests of e civil engineer.

On p. 10 of the Paper, the Authors had given details of the type of

concrete used in the tests carried out at Cardington. That concrete appeared to Mr Critchell to be a particularly rich mix, although in fact it was made with an aggregate which he would have thought was not particularly resistant to jet effects. He would therefore like to know if the study of the effect of jets on concrete had been carried further to assess the most economic type of mix to use on jet aircraft runways. Did the aggregate in fact have an effect ? He understood that igneous rocks and limestones had better resistance to heating effects.

On p. 5, the Authors had mentioned aromatics. The question of the composition of the fuel was important to the manufacturers of certain of the materials used on runways, in particular those concerned with joint sealing compounds. He had wondered how anyone had succeeded in producing a sealing compound which was resistant to aromatics, for neither bitumen nor tar would fit into that category. On looking further into the matter, he had found that the more active components of the fuel were not in fact aromatics but were cyclic aliphatics or alicyclic compounds. Those were unsaturated compounds, but not of the aromatic type ; they were therefore more or less intermediate in their properties between aromatics and aliphatics. They were more closely related to the paraffins and it was for that reason, therefore, that materials with considerable resistance to the new fuels could be obtained by the adoption of materials such as coal tar.

In the development of those materials, however, there had been a number of difficulties because they could be produced as either hot-laid or cold-laid materials. In view of subsequent heat treatment, the obvious choice was a cold-laid material which would harden by setting through some medium or other. On the other hand, initial experience with cold-laid materials had indicated that there were certain problems which were insuperable from an application point of view. For example, one could not get materials with a sufficiently long setting-time, or with a sufficiently well controlled setting-time, for aircraft to be able to run over the jointing without displacing it, as soon as the material had been placed. Similarly, cold-laid materials relied on water or solvents to soften them so that they could be placed, and on loss of those solvents or water they tended to shrink, which was, of course, a bad thing because they then tended to part from the concrete. He gathered, therefore, that most of the efforts had been made with hot-laid materials. These materials must be such that it should be possible to apply them to joints up to 3 inches deep, formed by the T-bar which was from $\frac{3}{8}$ inch to $\frac{1}{4}$ inch wide, so that the joint width could be anything from $\frac{3}{8}$ inch (or less) to $\frac{1}{2}$ inch. It was therefore necessary to produce a material that would flow to the bottom of a very narrow and deep cavity and that could be placed in one operation and would subsequently be resistant to jet blast.

He shared the Authors' doubt whether such a material was possible. He thought that some method of providing a layer over the material partly to insulate it from the heat, and particularly from the blast of the jet,

would be necessary. He could not visualize any hot-laid material being adequate both to resist the heat of the jet and also to be sufficiently " pourable " at a reasonable temperature to enable it to be placed in those deep and narrow joints.

In their test to assess the softening of the asphalt and tarmacadam carpets, the Authors had used a spike. Mr Critchell did not quite know how it was retained in position behind the jet, but it seemed to him that if the angle of the orifice was increased, then the vibrations set up in the spike arrangement were likely to increase the softening, or the apparent softening, by causing the needle to penetrate farther. He imagined, however, that test results had been correlated with full-scale experience, and that the apparatus did in fact give an estimation of the effect of heat rather than of blast on the softening.

Dr R. S. Millard said that, as the Authors had indicated, the problems of providing surfacings for use by jet aircraft seemed to arise mainly with bituminous surfacings. Concrete itself seemed to stand up quite well to the jet aircraft in service at the moment, and he thought that the relatively smaller problem of providing joint sealing compounds seemed to be well on the way to solution.

When the destructive effects of jet aircraft had first been appreciated, it was believed by some people that bituminous surfacings might turn out to be quite unacceptable, and indeed the continuing rapid developments of jet aircraft still made the future rather uncertain. He had read recently, in a report in an American journal,[5] that surface temperatures on the runway might ultimately reach a figure (exceeding that given by Mr Mann) of 1,600° F., or 870° C. Could the Authors give any indication whether such temperatures were to be expected, or did they believe that the situation would improve and that aircraft designers would become more appreciative of the airfield contractor's problems and do what they could to help ?

It might be that there were aircraft in prospect which were more damaging than were the Attackers, Vampires, and others, but with those at present in service it did seem that it was quite practicable to use bituminous surfacings of normal type over most of the area of the runways, and reserve those special, and almost certainly more expensive materials, for use where the effects of the jets and of fuel spillage were particularly severe.

With the renewed possibility that bituminous surfacings could be used, Dr Millard said that it was very desirable to establish what characteristics in a surface best fitted it to withstand the effect of jet aircraft. It seemed that most of the effort so far had been in developing new forms of surfacing. He suggested that there was a field for improving the ordinary type of surfacing by possibly quite minor adjustments in specification which might make much greater improvements to the performance of material

[5] " Navy holds Seminar on Jet Airfield Pavements." *Rds & Streets*, vol. 95, p. 64 (May 1952).

in proportion to the extra cost which they would involve. He indicated some of the general characteristics which ought to be considered.

The first was the use of aggregates of high thermal diffusivity and low conductivity, such as limestone. Some of the igneous rocks were also good as against flint gravel. Another characteristic was the choice of type and grading of the aggregate such that when the binder softened the material still retained some cohesion and internal friction, and was thus less readily displaced by the jet blast. A further point, to which Mr Mann had referred, was the use of a smooth surface texture. Dr Millard imagined that the best results would be achieved with a surfacing as smooth as considerations of slipperiness would permit.

Binders of as high viscosity as possible should be used. He had noticed in the Paper that the tests had been done on asphalts made with a relatively soft-penetration bitumen. He suggested that there would be less susceptibility to softening if harder binders were used. Bitumen had an advantage over tar in that respect, in that it was less susceptible to temperature changes; on the other hand, tar had a much superior resistance to attack by the fuels.

All the factors he had mentioned would probably have a considerable effect on the performance of the surfacing. Each of them involved possible modifications in composition which were very well within the scope of present methods of production. There were other points, such as the use of fibrous fillers or possibly light reinforcement, which had distinct possibilities and which again involved very little adjustment in the mixing and laying processes, and very little extra cost.

In the Paper, the Authors had given some very useful information in their results obtained with their captive jet engine on a limited number of surfacings. Each of those surfacings had been designed without any thought to its special jet-resistant qualities, and he thought it was most desirable to continue the work, possibly also using small model jets in the laboratory, to establish how bituminous surfacing mixtures should be designed so that they were best able to withstand the effect of jets.

There was one other factor which the Authors had mentioned briefly in the section on airfield lighting: that was the greatly increased tire pressures which had been proposed and which, he believed, were already in use in some military aircraft. Could the Authors indicate whether there was as yet any evidence on the behaviour of airfield surfacings under these higher tire pressures? He expected that the worst condition would be under a stationary aircraft. If there was a problem there—and he had not heard that there was, except in anticipation—it was another subject which he thought could very readily yield to systematic investigation.

Mr Alfred Goode stated that he was still of the opinion, which he had expressed in the discussion on Airport Paper No. 16,[6] that tempera-

[6] See ref. 2, p. 29.

ure effects were receiving undue prominence, especially when considering ivil airfields. It would appear from the Paper and from the film which he Authors had shown that the Vampire was the greatest offender from he temperature point of view, and it seemed a pity—at least, in the eyes of the pavement engineer—that it had been adopted as a basic trainer by the Royal Air Force. He had been happy to note at the 1953 S.B.A.C. Display that the latest fighters, such as the Swift, Hunter, and Javelin, and bombers, such as the Valiant, Vulcan, and Victor, and also the civil aircraft, such as the Comet and the proposed Vickers V.C.7, all appeared o have horizontal jets and at such a height that the temperature effects on the pavement might be considered negligible. If aircraft such as the Vampire were to be used on a particular airfield, then he felt that special measures were warranted to produce surfaces that would withstand the temperature effects, but those would be needed only on areas which were very small compared with the total area of the pavement.

Could the Authors state why brick had not yet been given a trial, or, if it had, why it was not mentioned among the alternative forms of pavement mentioned in the Paper? He had previously suggested it in the discussion on Dr Knight's Paper,[6] and Dr Knight had himself suggested stone setts as a possible solution.

Mr Goode believed that brick would also supply the answer to the problem of fuel spillage. The spillage effect was very serious on black top, since it became tacky and got carried into the aircraft, on to the tires, and into the undercarriage gear, etc. Had the Authors ever encountered any spillage effects on the taxiway? He believed that, particularly with certain American aircraft, the spillage effect had been noticed as being serious on bends in the taxiway, and fighters especially were prone to spill fuel when they were changing direction while taxying with full tanks. That brought an additional area into the problem.

He had spoken of the two main problems—temperature and fuel effects; but he was surprised to notice that the third effect of jets, which certainly cost more money to counteract on an airfield than any other, had been rather cursorily dismissed in one paragraph headed "Erosion." He was referring to the blast effect, which was not only serious at aprons and landings, but also alongside taxiways and even runways. He knew of one airfield in Great Britain where large aircraft were at present operating with the outer jet pods at a span greater than the taxiway width. That taxiway had to be swept free of pebbles and sand, at least after two aircraft had passed and very often after one. Even if the jets were inboard and were within the taxiway width, the effects were seen on all bends on the taxiway and at turn-offs to and within the dispersal areas. It should be borne in mind that the velocity of a jet exhaust might be anything up to 300 miles per hour; even 100 feet away from the jet quite a gale was blowing and in dry weather, at any rate, turf would not stand up to it and got burnt and blown off and became seriously eroded. It would seem that

large areas of specially treated ground would have to be provided alongside dispersal standings, warming-up aprons, taxiways, and possibly runways. It was possible with civil aircraft, where time was not so important, that more towing could be undertaken rather than moving the aircraft under their own power, but that certainly could not be considered with military aircraft, especially fighters. The construction might well have to be deep, for it could easily be imagined that with oiled earth or stabilized soil to a depth of 3 inches, it could be blown away.

The provision of blast-proof areas, whether of, say, 6 inches of lean mix concrete or hardcore, or stabilized soil covered with black top, would introduce other problems such as storm-water drainage, the provision of ducts for cables, and so on. The areas treated would obviously have to be kept to the minimum, but it was probably not generally realized that, on a large airfield, half a million square yards might have to be surfaced in addition to what was normally regarded as the paved area, so there was plenty of scope for research on the problem of counteracting jet blast. He asked the Authors whether any experiments had been carried out to counteract the effects of blast on what might be termed the margins.

He wished to correct a statement made by the Authors on p. 2 of the Paper. If reference were made to Airport Paper No. 3, it would be found that in 1946 the Authors of that Paper, in reply to the discussion, had mentioned the effects of jets on pavements and had hazarded the opinion that concrete rather than black-top surfaces would have to be provided where temperature effects were encountered.

Mr G. S. Cooper first mentioned that he had recently been moved from investigational work of the kind described in the Paper to a post in which he had been brought face to face with finding an immediate answer, even if only a temporary one.

In the particular Command with which he was concerned, there were, fortunately, only insignificant temperature effects, but the spillage effect was very serious. Many of the airfields which were being used had asphalt surfaces on the hardstandings, and quite a number of them had tarmacadam surfaces. It had not been long before there had been really severe erosion caused by the AVTUR and AVTAG spillage, and the most successful expedient that had been found was the application of a cement grout. That varied a great deal in its results. It was very difficult to lay down an exact specification, and the principal requirement was that it had to be very thin. If it were put down too thickly, a skin was obtained which could flake off and come loose. It was most successful on a fairly rugous asphalt surface, since the cement grout skin lay between the particles and obtained a fairly firm key. There had been one case where the grout had been placed much too thickly. It had formed a skin about $\frac{1}{8}$ inch thick, and it seemed that some water had been trapped underneath it which, under frost action, had loosened the cement grout skin; it had

broken up into little plates about 9 inches in diameter, and on one occasion one of those plates had taken off and all but decapitated the station commander who was standing near by !

The other striking thing which emerged from the Paper was the fact that, where bitumen was used as a binder, there would probably always be found the problem of resistance to solution by aromatic fuels. Bitumen was the accepted basis of road and pavement construction throughout the world, and to the best of Mr Cooper's knowledge there was no substitute known at present. It would be very interesting to see whether any substitute would be found, in time, which might compete both in properties and in cost.

Mr J. M. Watson asked the Authors if they had carried out any experiments with a bitumen-grouted stone surface. He knew of one runway at a station from which Vampires operated where the surface was hard limestone, about 2 inch nominal size, semi-grouted with bitumen, and blinded with chippings ; at a later date it had had a surface dressing of bitumen and sand to improve the surface. It had been possible to maintain the runway ends in service, without undue difficulty, for normal operating.

Referring to the section of the Paper which dealt with drainage difficulties, he said that there was no doubt about the difficulties which the use of kerosene as a fuel had introduced, because kerosene did not effectively evaporate at normal temperatures. It would be helpful to the drainage engineer in judging the extent of the problem in the future if he could know what fuel was likely to be used. About 70 per cent of the constituents of AVTAG or J.P.4. boil within the gasoline range, and therefore spilt fuel of that type should be considerably less trouble to separate from the surface water drainage. On the other hand, kerosene had advantages from the operator's point of view : it was cheaper (at present, at any rate), it probably had a slightly higher calorific value, in B.Th.U. per gallon, and it was very much safer, so much so that he did not think any civil aircraft at present were using anything other than kerosene as a turbine fuel. On that point he was open to correction, but that was his information. It therefore appeared that the drainage engineer might have to legislate for the general use of kerosene in the future and take steps to separate it from the surface water run-off from areas where spillage was likely to occur to any appreciable extent.

Mr T. V. Somerville, commenting on Mr Mann's remarks about the type of aircraft which the aircraft designers were likely to design in the future, said that he did not intend to prophesy, but he felt that there were fairly obvious trends in aircraft design. First, there was the point that had been made by Mr Goode, that engines were probably going to be installed in a horizontal position. Personally, he saw no reason why the aircraft designers could not install them in that way. Another possibility was the more extensive use in heavily loaded aircraft of after-burning, with

increased temperatures and also increased velocities, but the installation of the engines in the horizontal position might ease that aspect.

Mr Mann had also mentioned the possibility of there being extremely high temperatures if there were vertical jet engines. The only point he wished to make on that was that presumably those extremely high temperatures would only be local and, that being so, some special precautions could be taken.

The other possibility was rocket-assisted take-off for aircraft, about which he was rather more worried. In the past the practice had been to install those units at an angle in order that some of the blast should miss the aircraft structure. If that were to continue, there would be damage to the pavements, since the velocities and the temperatures of the jet from those units were considerably higher than from the aircraft engine.

Finally, he wished to stress a point which was not always realized by the airfield engineer or indeed by the aircraft designer, namely, that compared with the total cost of operating aircraft, the airfield costs—the concrete and hardstandings, etc.—were small in proportion, so that they could afford to spend quite a lot of effort on protective devices on the airfield without unduly increasing the total operating costs of the aircraft.

Mr P. J. M. Robinson asked the Authors if the deleterious effects of jet aircraft, which appeared, apart from one case of cracking in the concrete slab, to be associated with bitumen, would not be completely eliminated by the use of soil-cement. He had heard that soil-cement had a good performance with jet aircraft. Had the Authors tried that ?

Were the Authors able to say whether any research would be carried out on what the actual movements of the jet aircraft were ? The Authors had produced a Paper which contained all the information about what happened under jets ; would it now be possible to provide the designer with authoritative information on what parts of the airfield needed particular attention paying to them. For instance, how much spillage was there at servicing and maintenance points ? How much running-up was there at those points, and what was the fuel spillage to be expected at the corners of runways due perhaps to swilling ? He believed that civilian aircraft had to run-up their engines at the side of the runway before going on to it ; would special precautions have to be used there ?

He wished to ask Mr Cooper, who had expressed doubts about there being cheaper forms of construction than bituminous construction, whether he had considered cement stabilization in view of its better performance apart from its well-known cheapness.*

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* Mr Cooper, in reply to this question later, said that, in his previous remarks, he had been describing remedial measures on existing asphalt and tarmacadam surfaces, and would never consider putting soil-cement on top of that. He thought it had been fairly well established by tests that soil cement as an exposed surface would not stand up to weathering and jet effects. He believed the general opinion was that there had to be another more durable wearing surface on top of the soil cement.

Mr G. M. J. Williams observed that the Authors did not seem to be able to give a completely satisfactory solution to the problem of sealing the joints of concrete pavements, and he wondered whether they had considered the use of prestressed concrete pavements, which would require far fewer joints than the normal concrete pavement. He thought it would be possible to construct run-up areas and refuelling areas without any joints at all.

*** **Dr B. H. Knight** observed that, as one who in the past had had the privilege of seeing something of the work described in the Paper, he had found that contribution to technical knowledge of great interest. It was not, however, clear in the Paper whether any real solution to the problem of producing a heat-resistant black-top surfacing was in sight; he gathered that that was probably not the case at the present time, and he would like the Authors to amplify their conclusions on that most vital point.

He was also somewhat puzzled at the reference made on p. 21, which stated that " If all types of jet-propelled aircraft are to operate without restriction on the existing types of bituminous surfacings some form of protection against the destructive effect of the jet efflux will be necessary. . . ." It would appear that the question really was " How many destructive types of aircraft such as the Vampire are likely to use a given airfield ? " In other words, the airport engineers were in the hands of the aircraft designers as to what they might have to cope with in the future, and Dr Knight hoped that the Air Ministry would share fully the forward visions and ideas of such designers.

He was sorry that the Paper did not contain fuller details of the progress (if any) made in the matter of joint-sealing compounds for concrete runways. The mobile pouring unit shown in *Fig. 16* of the Paper seemed to be a neat contrivance, but he thought that difficulties were bound to arise from the obvious propensity of jet aircraft to expel jointing compounds from such runways. Had a satisfactory rubber-bitumen compound yet been found for such work ? He still regarded the problem of the jointing of concrete roads and runways as unsolved, but he might be wrong in that view. If so, he would like to be corrected by the Authors.

Mr R. F. Earley observed that the Authors had not mentioned the reciprocal effect of the pavement on the jet engine. He had recently asked one of the leading manufacturers of jet engines for his opinion on what was the greatest factor causing wear in such engines. The reply had been that, apart from mechanical and thermal effects on the engine structure (which could be reduced by research into problems of lubrication and metallurgy), the most serious trouble was caused by abrasion from grit, dust, and loose materials on the runway. It had been amply proved that

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*** This and the following contribution were submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

poor-quality concrete and materials such as " black top " (which were reduced to their primary constituents by blast and thermal action) should be avoided.

Mr Earley believed that, for the warming-up bays, only the best-quality dust-free concrete, without expansion joints, or even a pavement of dressed granite setts should be used. For large aerodromes, where jet engines were continually warming-up, there might well be justification for the expense of constructing an open metal grid covering a pit several feet deep. That would overcome the problems of both loose grit and oil spillage. It seemed likely that, whilst the discharge temperatures of commercial aircraft were likely to decrease, in order to reduce noise and fuel consumption, the discharge temperature of military aircraft (with reheat) was likely to increase considerably.

Mr Robertson, in reply, said that the concrete mix to which Mr Critchell had referred was richer than that normally used for concrete pavements, for in the case in question the slabs had been laid by hand and concrete of a rather higher workability had therefore been required to ensure full compaction. The types of jet aircraft at present in operation had no apparent detrimental effect upon high-quality concrete, so the Authors had not so far investigated the question of using special aggregates. If surface temperatures ever reached 500° C., however, aggregate would certainly have to be carefully selected ; at that temperature flint gravel, for example, rapidly deteriorated and would probably cause disruptive expansion. The spike mentioned in the Paper had been mounted on two lengths of steel tube which ran across the area of impingement of the jet efflux on the pavement and which had been fixed to the ground at each end. That device had given a good assessment of the overall effects of the jet stream on the surfacing.

He agreed with Dr Millard that much could be done to improve the resistance of bituminous surfacings to blast from jet aircraft by modifications to the aggregate grading and the use of fibrous fillers. Tests carried out on the second type of protective carpet, using a modified tar-pitch binder and described on p. 26, had confirmed that the addition of a fibrous filler increased the resistance of the carpet to the destructive effects of the jet efflux. However, surfacings based on bitumen as a binder would always be susceptible to the solvent action of the fuels.

Although higher-viscosity binders would undoubtedly tend to reduce the initial softening of the surfacing, it had been found in practice that there was little difference between the softening of asphalts using 180/220- and 60/80-penetration bitumen as binders.

He thought that a possible solution to the problem of vertical take-off might be to run the aircraft on to a grid with a large pit underneath, so that the jet stream would be directed downwards into the pit. As Mr Somerville had pointed out, areas requiring such treatment would be

omparatively small if the term " vertical take-off " was to be interpreted terally.

Mr Williams had suggested the use of prestressed concrete. A trial rea of pavement had been laid at London Airport [7] using that method of onstruction, and it had been found to cost considerably more than con- rete laid to the existing specification. In the present days of limited xpenditure, economic considerations would probably preclude the use of restressed concrete construction.

Referring to the comments made by Dr Knight, Mr Robertson thought hat a solution to the problem of providing a heat-resisting black-top urfacing was in sight, provided that certain practical difficulties could be vercome.

All rubber-bitumen joint-sealing compounds were attacked by jet uels and for that reason, as stated in the Paper, sealing compounds based n selected grades of coal-tar and pitch had been developed. The require- ments of joint-sealing compounds to resist the heat and blast effects of et aircraft were so conflicting that it was very unlikely that any com- letely satisfactory material could be produced. The present position as that compounds had been developed which appeared to be satis- actory in respect of adhesion and ductility at low temperatures, resistance o ingress of grit, and resistance to the solvent action of fuels. Such com- ounds were, however, displaced to some extent by the more severe types f aircraft such as the Vampire, although the displaced material remained acky for only about half a minute after the application of the jet efflux. ince those compounds were recent developments, no information was et available on their weathering properties.

Mr D. H. Carrack, who also replied, referred to Mr Critchell's well- nown work in connexion with joint-sealing compounds. With regard o the word " aromatics " Mr Carrack explained that the Authors, not eing chemists, had had a certain amount of difficulty in finding out the eaning of the word insofar as it related to jet fuels. For the benefit of vil engineers it might be of interest to point out that the term " aromatic ontent," which was used in the Paper in a general sense, was an expression sed by the petroleum industry and in fuel specifications to describe rtain types of unsaturated hydrocarbons.

He agreed with Dr Millard's remarks that a smooth surface texture ould give the best results with any material. However, arising from the gher landing speeds, there was a continual war between the surfacing ntractor who tended to lay a smooth-surface-textured asphalt and the perational side who, for obvious reasons, wanted a rough non-skid surface. aboratory tests with small-scale models of engines had possibilities, though they would scarcely replace full-scale tests. Dr Millard had ked about the behaviour of bituminous surfacings arising from the use

[7] " Prestressed Concrete Exhibition." *Rds & Rd Constr.*, vol. 27, p. 134 (Apr. 49).

of higher tire pressures. Mr Carrack thought it was generally agreed that stationary aircraft had the worst effect although he had noted " spewing " on a number of airfields where aircraft had to make sharp turns to manœuvre in and out of standings. More attention would have to be given to the stability of these surfacings.

He felt that Mr Goode had dismissed the heat problem rather quickly ; the fact remained that without the heat there would be no disintegration of bituminous surfacings from the blast, such as had been shown in the film. Nevertheless there was an existing problem to be dealt with. He quite agreed that, if aircraft like the Vampire were to operate, special measures were warranted.

The use of setts or bricks as a surface would raise the problem of forming a satisfactory seal at the surface. There would also be the important question of riding qualities to be considered and overcome, particularly in view of high landing speeds and tire pressures.

He regretted that more information on erosion had not been given, but the main purpose of the Paper, as indicated in its title, was to deal with the effects on airfield pavements. The associated problems had been mentioned with a view to giving a brief indication of the overall picture. The question of how the areas adjacent to taxi-tracks and terminal areas should be treated was one that required much more consideration. His own experience on airfields had been that it was very difficult to keep aircraft off any surface which looked like a runway pavement.

Mr Watson had mentioned bituminous grouted stone. Mr Carrack thought that probably all that would happen with that type of construction was that eventually the binder would flow, the resultant effect being rather the same as that shown for tarmacadam in the film. Mr Watson had also dealt with the question of fuels in relation to drainage problems. So far as he understood, civil aviation aircraft would continue to use AVTUR, which was a narrow-cut distillate and was commonly known as aviation kerosene, one of the advantages being that there was less fire risk involved and it was economical. The fuels used by military aircraft, however, were governed by the need for quantity, and a much wider cut distillate was therefore essential.

As to Mr Robinson's remarks on soil/cement stabilization as a solution to the problem, Mr Carrack rather regarded soil/cement stabilization as a purely temporary expedient before a permanent runway was built. He would, however, be very interested to hear where soil/cement stabilization had given a good performance with jet aircraft. Regarding Mr Robinson's questions about spillage, he thought that the spillage problem was quite clearly defined. Naturally the most spillage occurred on the standings and servicing areas, a certain amount occurred on the taxi-ways, but little or no evidence of spillage was ever seen on runways, unless for some reason an aircraft stopped on a runway and dumped fuel when the engine shut down, as had been shown in the film.

In presenting the Paper, the Authors had hoped that it would stimulate a discussion which would possibly result in a new approach to the subject. The various contributions to the discussion had given ample food for thought. He hoped, moreover, that the plight of the airfield engineer had been brought well and truly to the attention of the aircraft designers. It had been enlightening to hear Mr Somerville's remarks in dealing with certain aspects of aircraft design. Previously in his discussions with members of the aircraft industry he had been able to obtain very little direct guidance in this respect, particularly with regard to the future trend of aircraft design, except for statements to the effect that at present aircraft design was in a state of transition and the effects on runways, far from getting better, would get worse. It was therefore difficult for the airfield engineer to perceive what would have to be provided in the way of airfield pavements in the future, and until the position was clarified problems would have to be met and overcome on a day-to-day basis.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

27 October, 1953

Lieutenant-Colonel R. H. Edwards, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 24

"Overhaul and Repair of Lock Gates in the Port of London"

by

John Thomas Williams, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

One of the major maintenance tasks undertaken in the Port of London after the 1939–45 war has been the overhaul of a large number of lock gates and river entrances. The operations of stepping dry-docking and landing-lock gates, including the floating plant used and the general methods of carrying out the work by contract, are described.

Reference is made to the present-day condition of timber lock gates at the London Dock and the reasons leading to their replacement by all-welded steel ram-operated gates. The extent of corrosion on iron and steel gates, failures in gate structures, and the repair measures taken are reported.

The alterations made in spear-rod design on existing and new gates are recorded and a description is given of underwater repairs to existing pointing sills and roller paths using special methods. Notes are also made on the construction of limpet dams employed for the repair of bottom pivots and hollow quoins. Details are given of the work involved in replacing a large number of greenheart heel-posts and sills and the types of fixings used.

Particulars of the various entrances concerned are recorded and the cost of repairs to lock gates is analysed.

Introduction

In 1948, following the Chief Engineer's report on the condition of the river entrances and lock gates, the Port of London Authority approved a 4-year programme for carrying-out repairs which had been unavoidably deferred during the 1939–1945 war. Under normal circumstances, it is considered advisable to overhaul lock gates at intervals of not longer than 15 years; but at the end of the war many lock gates in the port had been in service for much longer periods and were, as a result, in need of extensive overhaul. In 1932, sixty pairs of gates were in use,[1] but through war

[1] Asa Binns, "Recent Developments in the Mechanical Equipment of the Port of London Authority." Proc. Instn Mech. Engrs, vol. 122, p. 575 (May 1932).

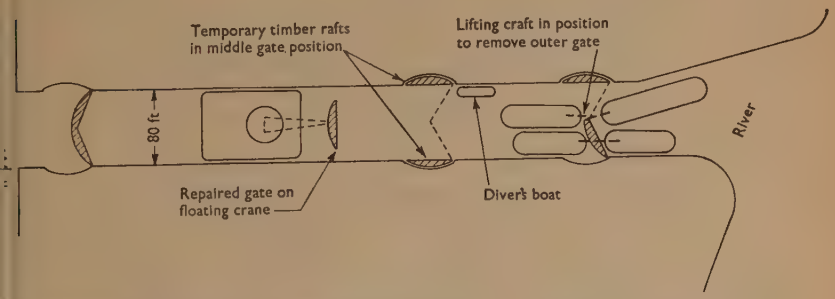
damage and other causes fifty-six pairs of gates were included in the programme (see Table 2*). The programme was arranged to suit the condition of the respective entrances and their importance from an operating point of view, bearing in mind the staff and labour position. Between 1949 and 1953, twenty pairs of gates have been overhauled and the construction of eleven new pairs of gates has been put in hand. One pair of new all-welded steel gates weighing 350 tons has been installed at the King George V entrance to the Royal Docks.

The 4-year programme also provided for some of the lock entrances, including pointing sills and lock-gate machinery, to be repaired simultaneously with the overhaul of the gates concerned. The cost of the work involved was about £600,000 before the addition of the rises in labour and material costs which have occurred since 1949.

It was decided to maintain three pairs of lock gates at all the important entrance locks, since the middle gates could then be regarded as spares for use in an emergency and during repairs to the other pairs of gates.

In the Royal group of docks, where the impounded level is maintained at a constant level of 2 feet 6 inches above Trinity High Water, there may be as many as fifty large cargo vessels berthed at any one time. As a precaution against the very heavy shipping losses which would be incurred in the event of serious damage to the gates caused, for example, by a ship collision, it was decided to provide a fourth pair of gates. Thus one pair of gates is kept in a vertical position at the head of one of the dry docks, with the timberwork boxed in for protection, for emergency use.

Fig. 2


Temporary timber rafts in middle gate position
Lifting craft in position to remove outer gate
80 ft
River
Repaired gate on floating crane
Diver's boat

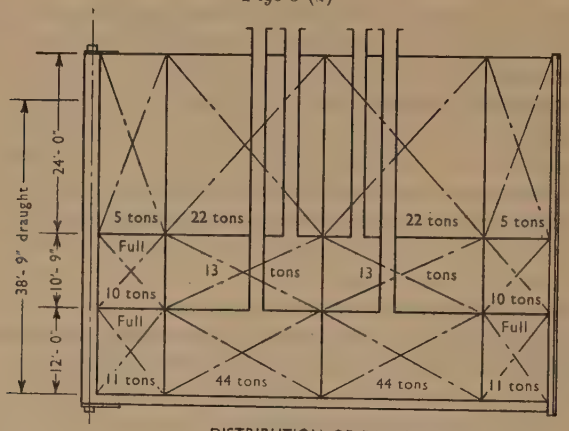
Typical Lay-out of Craft for Stepping and Unstepping Lock Gates

In the case of the larger entrances, where the gates are of considerable size, such as at Tilbury and the Royal group of docks, the gates have been floated into dry dock, in both the vertical and horizontal positions. At Tilbury it was possible, by obtaining complete possession of the dry dock,

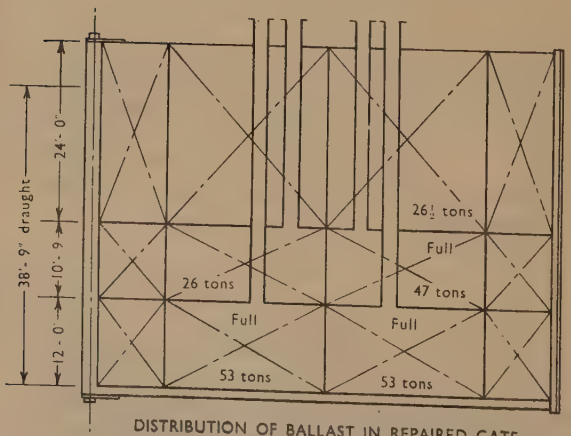
* Tables 2 and 3 are printed as an Appendix and will be found on pp. 86, 87.

to dock the gates horizontally on to specially prepared blocks (see *Figs 1* and *2*) and to complete the overhaul, without interruption by shipping calls on the dry dock, in the comparatively short period of 4 months. In the case of the Royal Docks, the repairs took much longer to complete because the gates were kept in the vertical position (*Fig. 3* (*c*)) to make the dry dock available for shipping, with consequent interruptions through the flooding of the dry dock. At other docks, the gates were lifted out

Figs 3 (*a*)



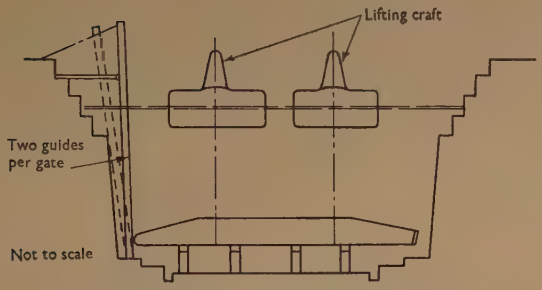
DISTRIBUTION OF BALLAST
PRIOR TO UNSTEPPING, 21.1.52
TOTAL LOAD ON LIFTING CRAFT: 105 TONS



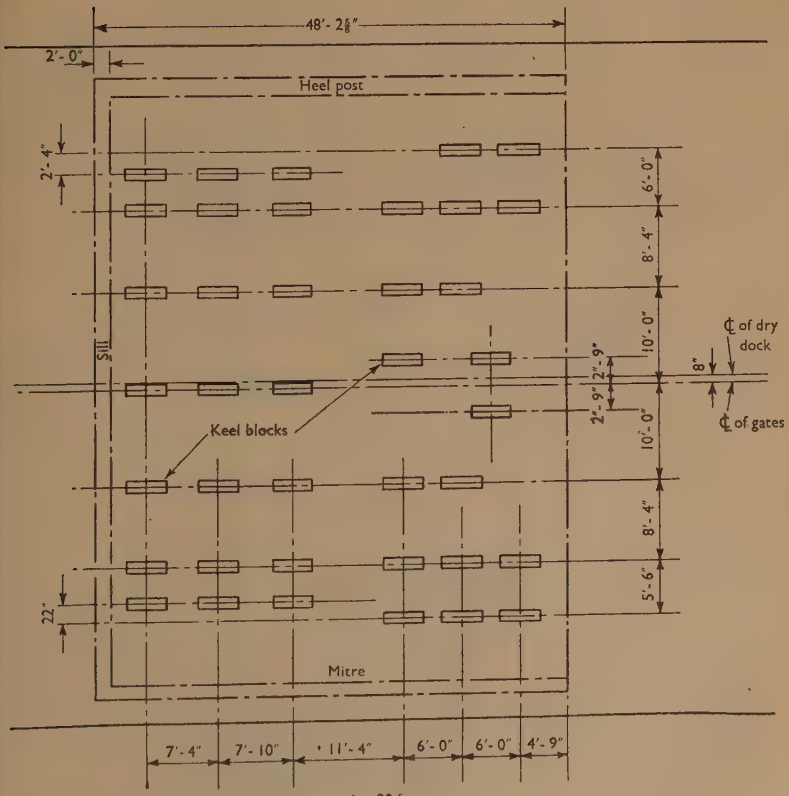
DISTRIBUTION OF BALLAST IN REPAIRED GATE
PRIOR TO STEPPING BY FLOATING CRANE, 8.5.53
TOTAL LOAD ON FLOATING CRANE: 102 TONS

TILBURY—DISTRIBUTION OF BALLAST FOR UNSTEPPING AND STEPPING 330-TON GATES AT WESTERN ENTRANCE

Figs 3 (b)



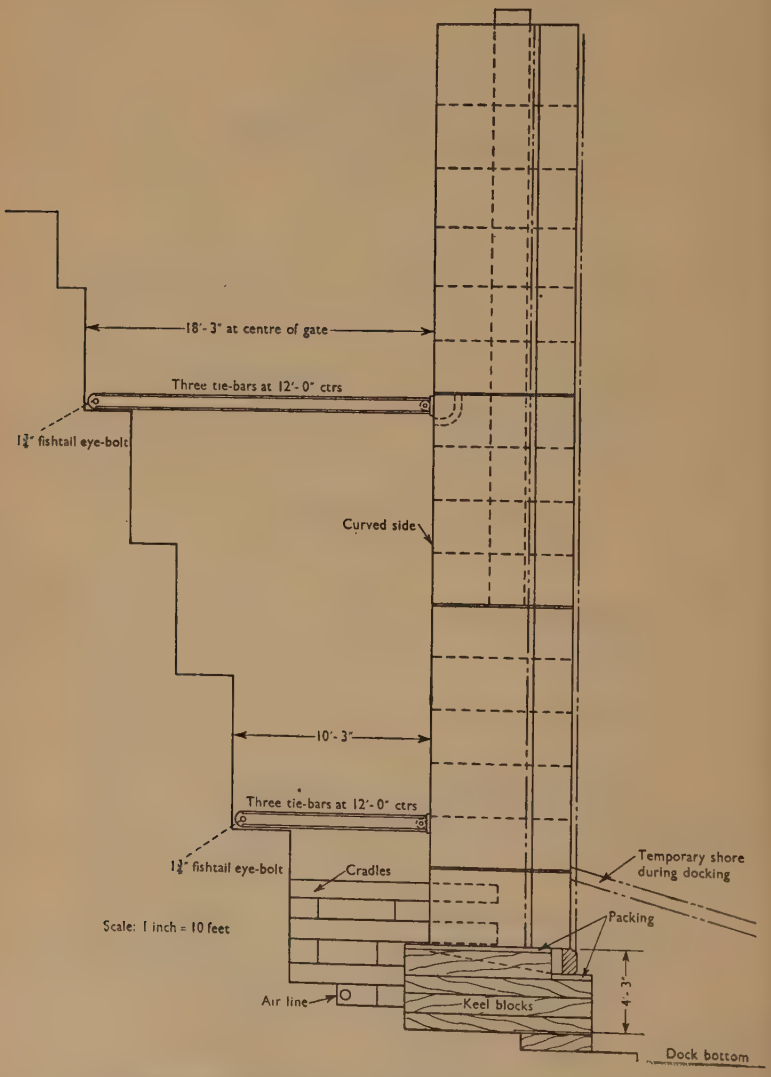
SECTION THROUGH DRY DOCK SHOWING METHOD OF POSITIONING GATES OVER BLOCKS



TILBURY UPPER ENTRANCE GATES. DOCKING PLAN

on to specially prepared sites by means of the Authority's own floating cranes and salvage craft. When the gates had been landed and cleaned of as much mud as possible they were thoroughly surveyed and an approximate schedule of repairs was drawn up by the Divisional Engineer in collaboration with the Authority's maintenance contractor, who undertook

Figs 3 (c)



King George V Lock. Gate Docked in Upright Position

the repair work in accordance with detailed orders by the Engineer. After the removal of all fittings, scaling, and the stripping of the timber heel-posts, sills, and mitres which were in most cases all renewed, closer examination was possible and supplementary lists of repairs were made out. Although these supplementary lists of repairs were in some cases extensive, the gates were generally found to be in remarkably good condition when it is borne in mind that the majority of those dealt with had been in service without repair for more than 20 years, including the war years.

Since it is usual for gates to be installed in a number of different positions during their lifetime, all the repaired gates have been fitted with a numbered plate to assist in identification and to enable a future history of each gate to be recorded.

Handling and Transfer of Gates

One of the following methods has usually been employed in handling the gates :—

(1) Controlled flotation. This was rarely used owing to the doubtful condition of the buoyancy compartments of many gates after a long period in service.

(2) Wreck-lifting lighters, of which the Port Authority have a number capable of lifting 100 tons each.

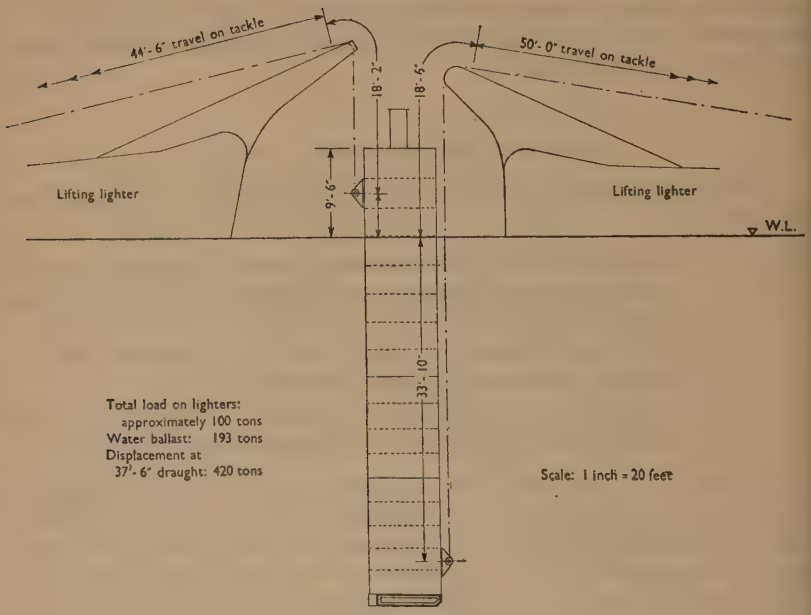
(3) Floating cranes of up to 150 tons capacity. The 150-ton floating crane is capable of stepping the largest gates in the port.

In the case of locks fitted with two pairs of gates, a floating crane was generally used to unstep the gates, but where middle gates occurred these were repaired first and the lifting lighters were used to unstep the next gates to be repaired, so that the floating crane could substitute immediately (on the same day) the newly repaired gate with the least delay and minimum interruption to the use of the lock. When lifting lighters were used, wire slings were reeved round or fixed to the gate (*Fig. 4*) and temporary timber struts were placed in the upper compartments, where the platework is of comparatively light section, to prevent damage from the concentrated load applied by the wires (*Fig. 4*).

A considerable amount of traffic is handled through the main river entrances every 24 hours and, since the period covering one tide was usually required to carry out any of the above operations, a detailed programme was prepared well in advance to suit the local tidal conditions. The operations were broadly similar in all cases, and those concerned with the handling of the upper entrance lock gates at Tilbury may be taken as typical. This entrance is 994 feet long and 110 feet wide, with 45 feet 6 inches of water over the centre of the sill. Each leaf of the double-skin steel tank gates weighs 330 tons. To save time on the day of the change-over from

the existing to a repaired gate all anchorage connexions were unscrewed and rescrewed beforehand. Wire " messengers " were also reeved beforehand for fixing the main lifting cables. The anchorage straps, etc. were usually removed by a 2-ton mobile crane, the operating ram and crocodile girder having been removed previously. The gates were also cleared beforehand of as much mud as possible and wooden plugs were inserted in the scupper holes of the tidal chambers so that the latter could be used for additional buoyancy if required. The repaired gate was ballasted at

Fig. 4



TILBURY WESTERN ENTRANCE. 330-TON LOCK GATE. ARRANGEMENT OF TACKLE FOR LIFTING AND DOCKING

the commencement of the operation to give about 90 tons weight on the crane with adequate clearance under the gate to prevent damage during manœuvring. For convenient handling as the gate was lowered into the water, the preponderance of weight over buoyancy was not allowed to fall below about 10 tons, the water ballast being supplied to the gate through fire hoses attached by instantaneous connexions for convenience. Some gates were fitted with 6-inch-diameter flooding valves operated by extension spindles, which reduced the time required for ballasting. The ordinary type of hydraulic ejectors employing flexible connexions were also used for pumping water out of the gates. Small portable air-pumps of much greater capacity suitable for lowering through the gate access

runks can be obtained, but their use necessitates an additional compressor, unless a local air-supply is available. Under normal conditions, where a hydraulic supply is available, the 2-inch-diameter ejectors are quite adequate and give trouble-free service in the gate under operating conditions.

Gates which had been repaired on the flat ashore were launched by the floating crane, using slings reeved round the gate, and then ballasted until they were in a vertical position, the crane slings being transferred to the lifting lugs provided on the gate. The timber sills were protected from damage by specially constructed shoes (*Fig. 5*) through which the wires were reeved. Where the bottom pivot is of the loose type it is liable to be displaced as the old gate is moved away from the quoin, and for this reason the diver was always sent down to ensure that the pivot was in its correct position before attempting to step the repaired gate.

The whole operation of removing the upper outer gate from the western entrance to Tilbury docks to prepared blocks in the dry dock, and stepping repaired gate and connecting it up to be operated temporarily by wires and capstans, was completed between the hours of 7 a.m. and 6.15 p.m.

To facilitate stepping and to ensure that the gate pivot-bearing fitted easily over the pivot, the gate was trimmed down by the mitre end about inches. This was arranged by making the slings at the mitre end about inches longer or by inserting an extra shackle. Once the gate was stepped properly on the bearing the gate was restored to a vertical position by slewing the jib of the floating crane or by additional ballast at the heel end.

When several months elapsed between the removal of gates and their replacement, as in the case of middle gates or entrances having only two pairs of gates, a timber raft made of 12-inch-square or similar-sized timbers was placed in the gate recess to protect the hollow quoins from damage by shipping. The raft was able to rise and fall with the water level by means of a chain passing through a steel-plate fairlead. The chain was fixed at cope level and to a 50-cwt cast-iron kentledge block on the bottom of the lock.

When the floating crane was handling large gates (such as those weighing 350 tons) in the outer position, the programme was arranged to ensure that the gate was stepped at or near high water and before the tide ebbed to a level where the loss of buoyancy caused the load on the crane to approach the safe working limit.

The medium-sized gates of approximately 150 tons weight could be moved by the wreck lighters and lifted just clear of the water in a horizontal position before transferring them to the floating crane for landing ashore. It frequently happened, however, that the gates contained such a large quantity of mud that they were beyond the crane's capacity, which is less than the combined lift of the four lighters. A number of selected rivets were then burnt out, and the mud was washed

out through the rivet holes by pressure hoses. Owing to the inflammable nature of the gases present in the river mud and silt, tests were made to ensure that the gates were gas-free before applying burners or permitting men to enter the tank chambers.

When handling large tank gates, care was taken to ensure that the ballast was distributed as far as possible so that the tanks were either full or empty, in order to keep the quantity of " free " water to the minimum and avoid sudden variations of load on the lifting tackle.

The typical lay-out of the craft employed for replacing an outer gate by a repaired middle gate is shown in *Fig. 2*. During the stepping of the gate, the lifting lighters were moved clear of the lock and " parbuckled " the removed gate to a horizontal position. In this position it can be safely " locked in " to the dock at a lower tide level, which may be necessary if the stepping operation is delayed by some unforeseen incident.

Condition of Gates

By far the oldest gates, all constructed of timber, are to be found in the London and St Katharine Dock system, which has three river entrances—at Wapping (40 feet wide), St Katharine Dock (45 feet wide), and Shadwell (60 feet wide). The first was closed in 1940 for economy reasons and the gates were strutted in position.

The condition of the St Katharine Dock gates deteriorated during the war and the pointing sill was damaged so that the gates leaked badly. The gates were hand-operated by chains, but following repairs to the pointing sill they were made to mitre satisfactorily pending the delivery of new gates which were ordered in October 1952.

The new gates are to be all-welded ram-operated tank gates with direct-operating hydraulic-ram machinery and hydraulic sluices. Because of the greater moulded width of the new gates, it will be necessary to widen the gate recesses by 9 inches to accommodate the sluice machinery.

Three pairs of all-welded gates have also been ordered for the Shadwell entrance lock, although the present timber gates, installed in 1855, are in very good condition. The main horizontal ribs and vertical struts in these gates were built of oak and were perfectly sound when inspected recently. The oak heel-posts and mitre-posts were scored and worn to some extent, but were otherwise sound. The hand-operated sluices and tie-bolts were in fair condition and the only signs of real decay and damage were on the timber sheathing and fendering, which are exposed between wind and water and come into contact with moving ships and barges. Some of the ironwork, chains, and brackets also required renewal.

It was decided to install new gates in order to increase the effective available depth of water at this entrance by substituting direct-operating rams for the chains and also raising the level of the machinery. The

existing pointing sills at the St Katharine entrance are curved in plan, but the new gates are being designed flat on the pressure side.

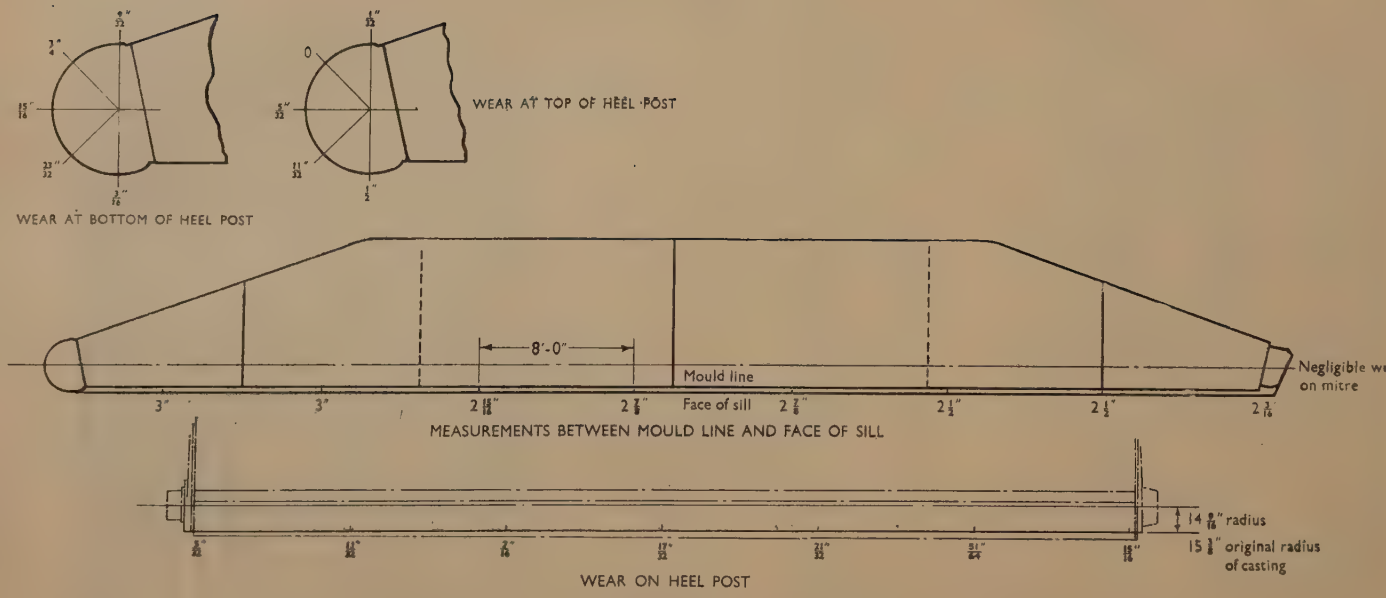
On most gates which had been in service for a long period, the greenheart heel-posts showed considerable signs of wear (see Table 1). In

TABLE 1.—WEAR OF GREENHEART HEEL-POSTS.

Gates	Method of operation	Time in service: years	Entrance dimensions	Radius of heel-post: inches	Maximum wear: inch		
					Top	Mid-way	Bottom
Tilbury—Upper entrance							
Outer upper					$\frac{3}{8}$	1	$1\frac{13}{16}$
„ lower					$\frac{3}{16}$	$\frac{7}{8}$	$1\frac{9}{16}$
Middle upper					$\frac{5}{32}$	1	$1\frac{15}{16}$
„ lower	Hydraulic ram	23	110′ 0″ by 45′ 6″	$16\frac{1}{2}$	$\frac{1}{4}$	$\frac{17}{32}$	$\frac{7}{8}$
Inner upper					$\frac{3}{16}$	$\frac{5}{8}$	$1\frac{1}{16}$
„ lower					$\frac{5}{32}$	$\frac{5}{8}$	$1\frac{1}{16}$
Surrey—Greenland entrance							
Inner upper	Ram and crocodile girder	20	80′ 0″ by	$13\frac{1}{2}$	$\frac{9}{32}$	$\frac{5}{16}$	$\frac{9}{16}$
„ lower	Hydraulic power		35′ 3″		$\frac{5}{32}$	$\frac{3}{16}$	$\frac{3}{4}$
Surrey lock							
Inner upper	Chain	16	50′ 0″ by	12	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{8}$
„ lower	Hydraulic power		27′ 3″		$\frac{3}{8}$	$\frac{3}{8}$	$1\frac{3}{8}$

some cases as much as 2 inches of timber had been worn away. The wear usually varied from maximum at the bottom of the heel-post to minimum at the top and also varied round the circumference of the heel in the manner shown in *Figs 6*. For the larger entrances, it is usual for the bottom gudgeon of the gate to have a 1-inch clearance all round the pivots, so that more than about 1 inch of wear on the heel-post leads to wear of the bottom casting and granite quoin as shown in *Figs 6*. It has also been found that wear of the heel-post in excess of the clearance allowed in the bottom gudgeon bearing causes damage to the hollow quoin by projecting rivet heads in the heel-chamber seams. This would not arise if flush-headed rivets or welding were used. Typical examples of the wear found on various gates are given in Table 1. The wear on the mitre timber is usually small, but sill timbers have suffered severe damage by crushing, and the supporting angles have usually required stiffening against buckling. Typical examples of this kind of damage are shown in *Fig. 7*. The support to the clapping sill has also been strengthened

Figs 6



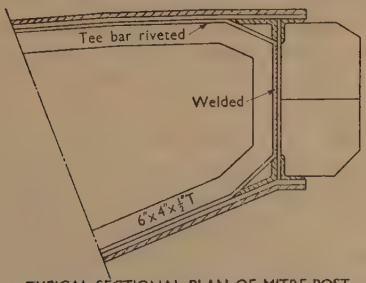
TILBURY WESTERN ENTRANCE. LOWER MIDDLE GATE. TYPICAL WEAR ON GREENHEART

in some cases by an additional angle, where this had been omitted, fixed to the sill back-plate to box-in the lower edge of the sill timber. Where gusset brackets supporting the clapping sill are fixed to the underside of the bottom tank compartment, it has been found that wasting of the angles occurs. This is usually most noticeable towards the mitre ends of the gates and appears to result from the intensified effect of eddies caused by the movement of the gate and possibly accelerated by chemical action from impurities in the dock water. For example, a $\frac{7}{8}$-inch-diameter bolt on a sluice door has been reduced to $\frac{1}{4}$ inch in about 12 months.

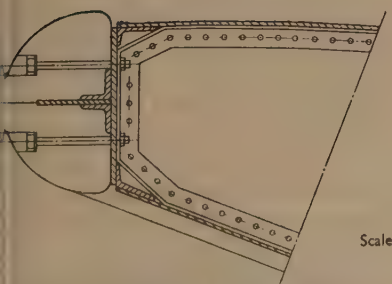
Gate Structure

In the course of the repair programme, note has been made also of the apparent weaknesses in gate structures and points where the design could be improved. First of all, it should be borne in mind that the loads applied to the gate structure, anchorages, and machines in actual practice are largely unknown. On several occasions, for example, the cause of a gate failing to come up to the sill has been traced to a waterlogged bale of

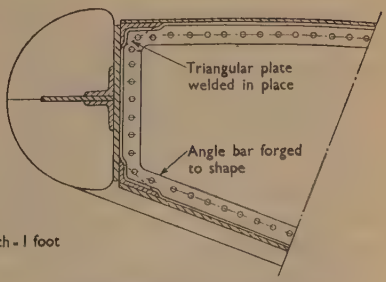
Figs 8 (a)



TYPICAL SECTIONAL PLAN OF MITRE-POST IN TIDAL CHAMBER SHOWING STIFFENING



TYPICAL SECTIONAL PLAN OF HEEL-POST IN TIDAL CHAMBER SHOWING ORIGINAL CONSTRUCTION



Scale: $\frac{3}{4}$ inch = 1 foot

MODIFICATION OF STRUCTURE IN TIDAL CHAMBER

King George V Lock Gates, Modifications to Mitre- and Heel-Posts

Esparto grass (about a 4-foot cube in size) or a waterlogged baulk of timber, trapped between the sill and the gate. The impact with an obstruction of this type when the gate is being closed under power must cause crushing of the sill timber and strain the gate structure generally.

In one case, it was found that, as a result of the horizontal deck angle-frames not being joggled to fit closely into the back plate, the rivets in the vertical seam adjacent to the heel-post became slack and leakage occurred. Since temporary repairs by reaming the holes and fitting steel bolts had been ineffective, the frames were cropped and joggled to the back plate and vertical angles. The deck plates were cropped back 12 inches and partly renewed to fit the corners at vertical angles. The plates on the non-pressure side adjacent to the heel-post were cropped and partly renewed where the contact face for the vertical frame had been eroded by the movement due to the failure (*Figs 8*).

Owing to gates drooping as a result of incorrect roller adjustment or heel-post wear and working in a " heavy " condition, the skin plating has sometimes failed in the top strake, which is usually a comparatively light section (about $\frac{3}{8}$ inch thick). In the course of repair the plates were partly renewed and reinforced by a gusset bracket midway between the decks in the top ballast compartment (*Figs 8*).

Another form of trouble in the ballast compartments is caused by the scuppers which, if of too small a bore and incorrectly positioned, direct water on to the stringer plate of the skeleton deck, which diverts it and causes wasting of the platework in the vicinity. This was dealt with by doubling the trunkways with a welded patch, the voids being filled with red lead by force pump. The butt cover plate of the access trunk was also welded at the lower and upper edges in chamfered form to reduce eddying. These defects could probably be avoided by fitting a large scupper pipe instead of the 90-degree-bend type and forming a modified bellmouth orifice direct through the shell-plating. Provision would also be made for blanking-off during handling of the gate for repairs.

There is a tendency for the tidal chambers to trap an excessive amount of mud, particularly in inner gates which work at a constant water level, because the scupper holes are usually situated on the pressure face some distance above the watertight deck. To remedy this the scuppers have been transferred in some cases to the non-pressure side a few inches above the watertight deck. It is hoped that the resultant increased scouring action caused by the rise and fall of the river and lock level will help to keep the gates clear of mud and reduce the load on the anchorages. The weight of mud in a gate may amount to as much as 10 per cent of the weight of the gate.

Serious wasting of the platework has also occurred in the gate sluice trunkways (see *Fig. 9*). Grooves $\frac{3}{8}$ inch to $\frac{1}{2}$ inch in depth have occurred in the side and bottom plates and up to $\frac{1}{4}$ inch in the top plating. The leading and discharge edges of the plating, which was normally $\frac{7}{8}$ inch

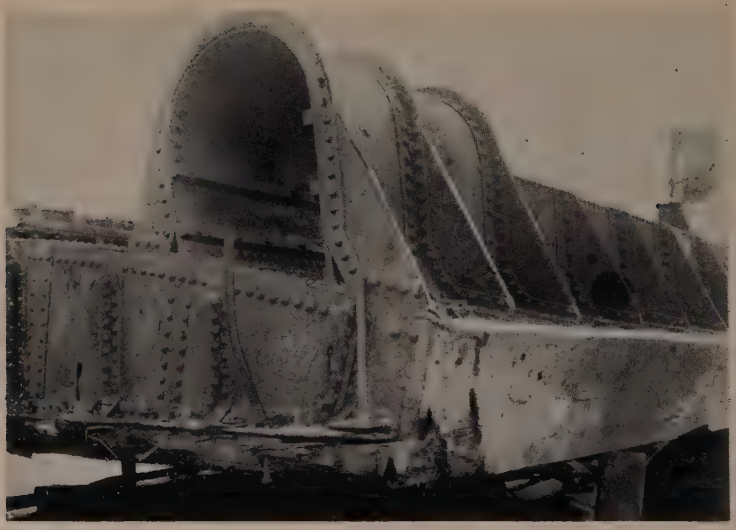
Fig. 1

TWO 330-TON LOCK GATES IN DRY DOCK AT TILBURY

Fig. 5

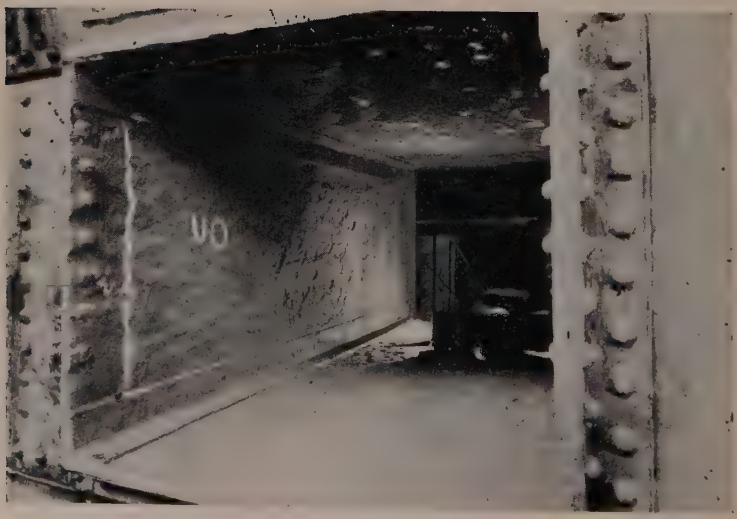
GATE AT TILBURY, SHOWING GREENHEART PROTECTING SHOES AND RUBBER ANGLE FIXED TO SILL

Fig. 7



Typical Damage to Lock Gate Sill

Fig. 9



King George V Dock. Grooving in Sluice Trunk

Fig. 14

Surrey Docks Basin. Bottom Pivot and Repaired Hollow Quoin

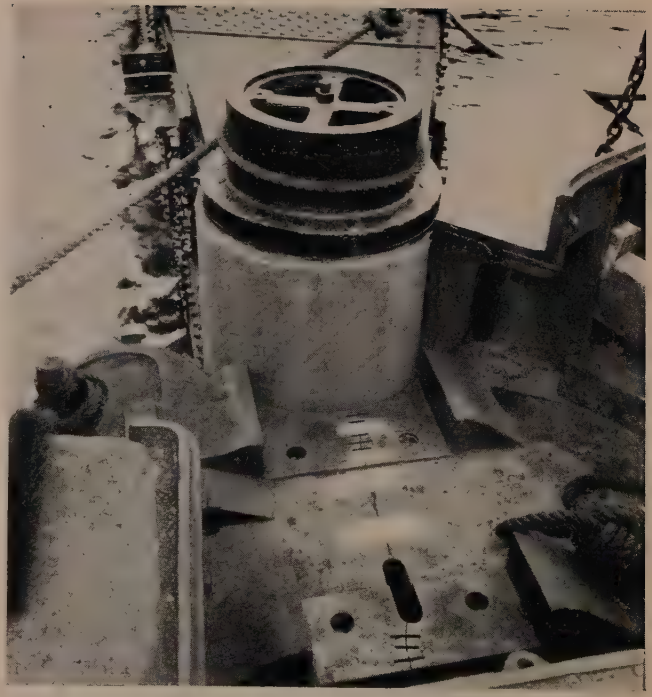
Fig. 18

Plan View of Limpet Dam in Position (Surrey Docks)

Fig. 22

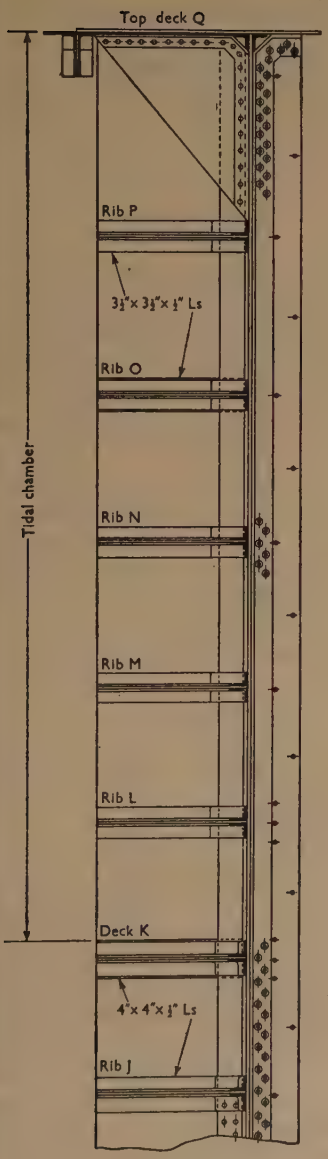
TEMPLET FOR DETERMINING EXACT LENGTH OF GATE

Fig. 25

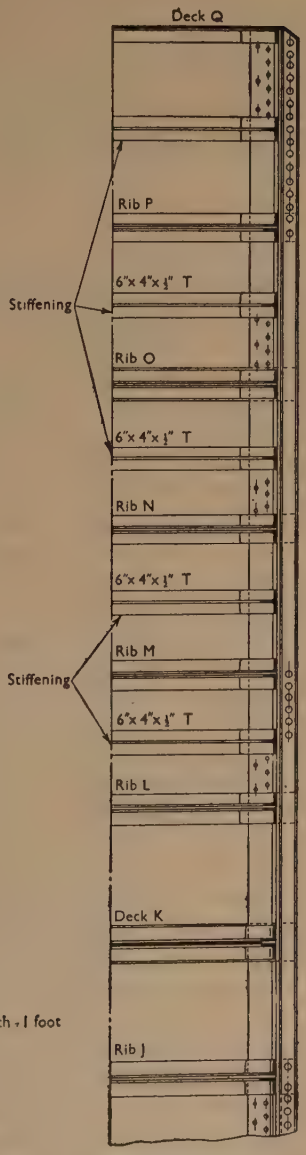


SURREY DOCKS. REPAIRS TO DAMAGED ANCHORAGE CASTING

Figs 8 (b)



SECTIONAL ELEVATION OF HEEL-POST



Scale: ½ inch = 1 foot

SECTIONAL ELEVATION OF MITRE-POST

KING GEORGE V LOCK GATES. MODIFICATIONS TO MITRE- AND HEEL-POSTS

thick, were badly grooved between the rivets. It was observed that the mitre side of the sluice trunk nearest to the heel-post was most affected, probably owing to some combined effect of the gate and the land sluices. These trunkways have been repaired by fitting $\frac{1}{2}$-inch plate liners welded on all edges, any voids being force-pumped with red lead. The leading and discharge edges were built up by welding to skin plates at 45 degrees to reduce the possibility of eddying.

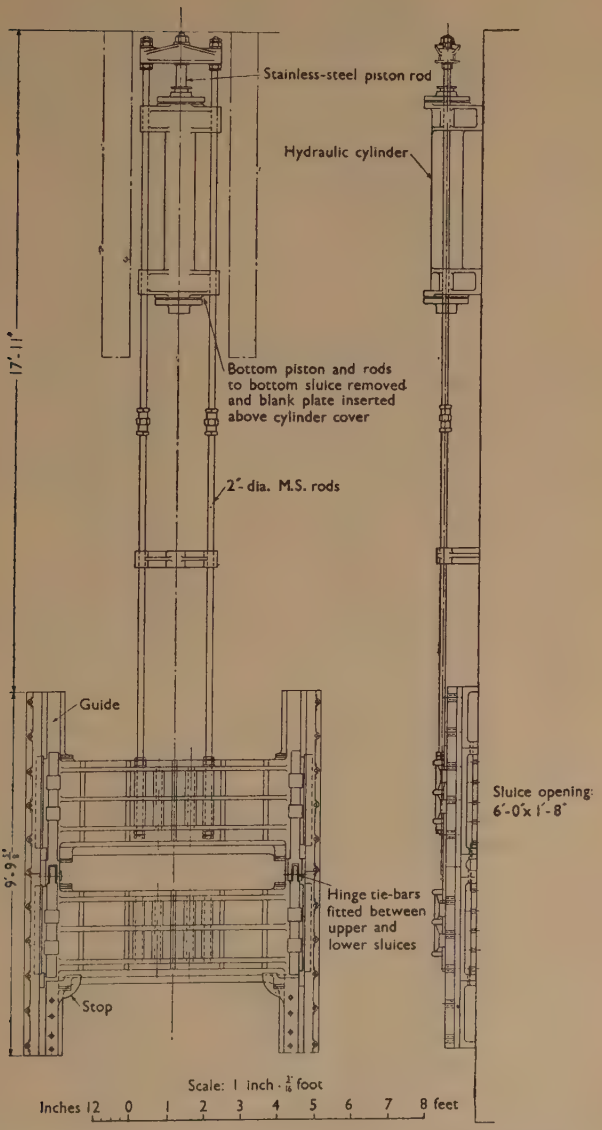
Iron Gates

The condition of the gates at the Surrey Entrance and Basin of the Surrey Commercial Docks was also noteworthy in view of their age. The two pairs of basin gates and inner lock gates were constructed of iron in 1858 and are chain-operated. This entrance is operated in a similar manner to the Shadwell entrance, London Dock, the larger vessels being locked in and out with the aid of the basin, since the lock itself is only 250 feet long.

At the end of the 1939–1945 war, the inner gates were leaking badly and the lower gate pivot was loose. In preference to taking these gates out first and leaving the entrance protected by the outer gates only, the basin gates, which were not operational as a result of deferred maintenance, were made first priority. They had been earmarked for replacement during the war, but it was decided, in view of the steel position, to lift them out for examination to see if they would serve for a further period after reasonable repair. This was done during one of the busiest periods of gate repairs undertaken at the Surrey Docks, when one pair of 165-ton gates at the main Greenland entrance were replaced and the basin gates lifted out and landed, eight gates being handled in all in the course of 1 week. The basin gates, though 95 years old, were found to be structurally sound with practically no wasting of plates.

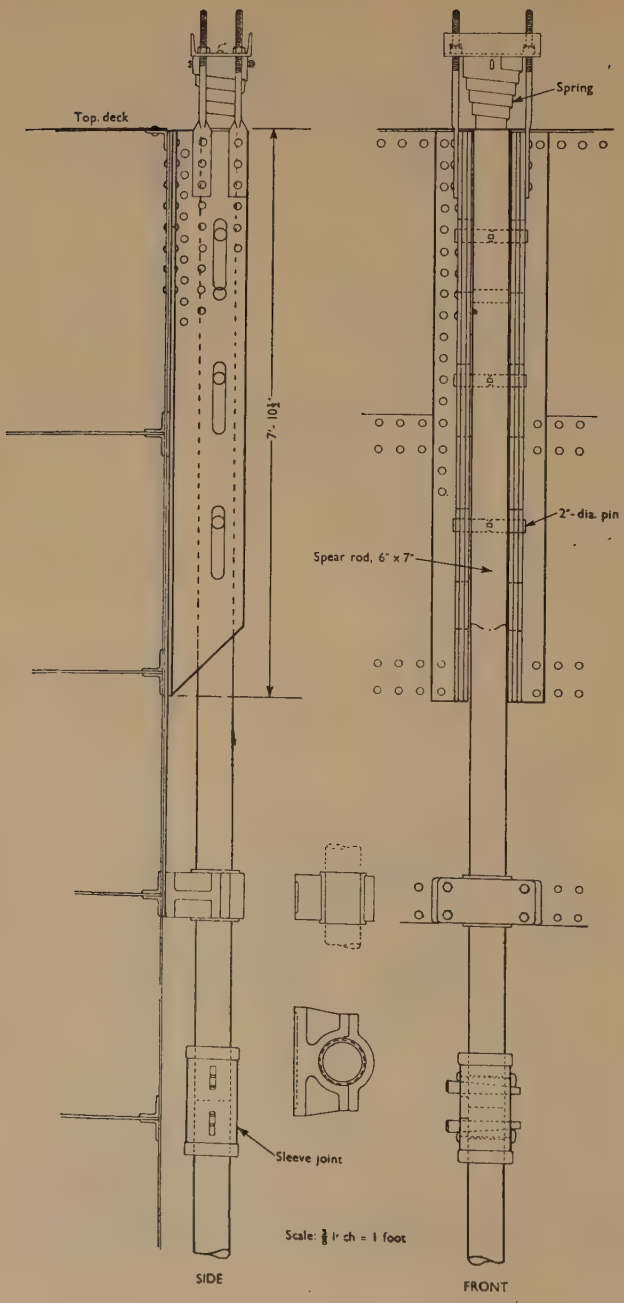
It was observed that all the main lower plates (originally $\frac{1}{2}$ inch thick) were in good condition, although some deterioration showed on the two upper strakes (original thickness $\frac{3}{8}$ inch), there being a full $\frac{1}{32}$ inch depth of hard black scale on the broken faces. Test pieces were taken from this area, which showed a strength of 20·2 tons per square inch along the grain and 17·3 tons per square inch across the grain. When bent over a $\frac{15}{16}$-inch-diameter former the plate failed at 29 degrees. The greenheart sills and heel-posts, however, were badly worn—up to nearly 2 inches in places—and the hand-operated gate sluices were so corroded as to be useless. Since the condition of these gates did not justify scrapping, they were repaired and the hand sluices replaced by hydraulic machinery incorporating stainless-steel piston rods. The double sluices on the inner gates at the Surrey Lock were modified to single-piston operation to avoid automatic opening of the sluice by a leak or failure in the hydraulic power system (*Figs 10*).

Figs 10


Stainless-steel piston rod
Hydraulic cylinder
17'-11"
Bottom piston and rods to bottom sluice removed and blank plate inserted above cylinder cover
2" dia. M.S. rods
Guide
9'-9½"
Sluice opening: 6'-0" x 1'-8"
Hinge tie-bars fitted between upper and lower sluices
Stop
Scale: 1 inch · 1/16 foot
Inches 12 0 1 2 3 4 5 6 7 8 feet

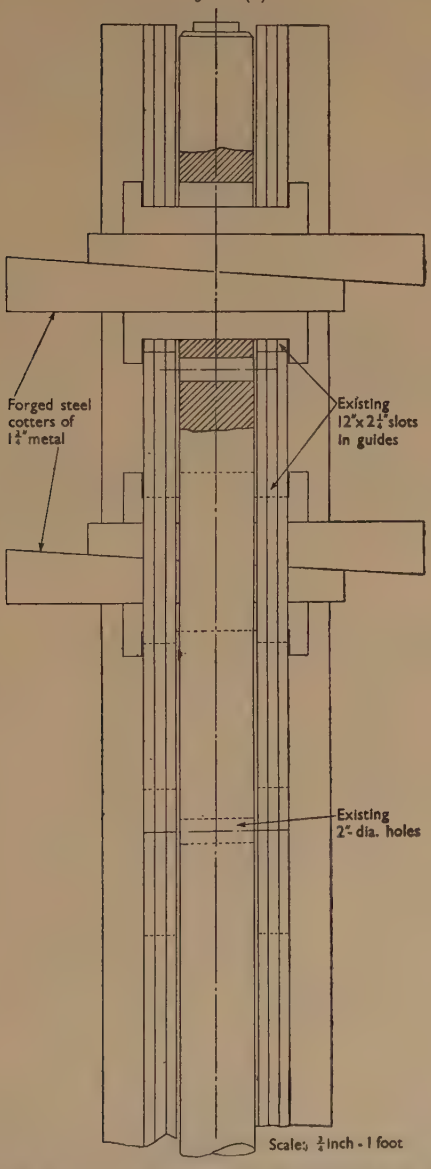
SURREY ENTRANCE. INNER LOCK GATES. MODIFICATIONS TO SLUICE MACHINERY

Figs 11 (a)



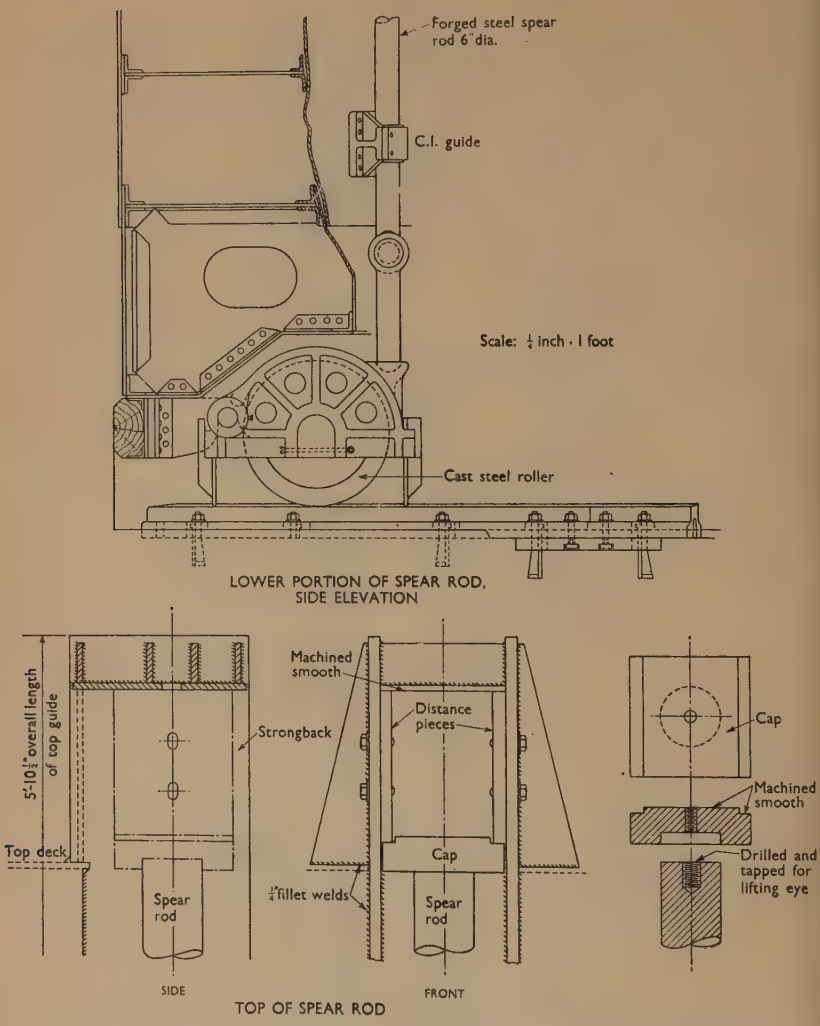
KING GEORGE V LOCK. SPEAR-ROD ADJUSTMENT

Fig. 11 (b)



KING GEORGE V LOCK. MODIFIED TOP PORTION OF SPEAR ROD

Figs 11 (c)



King George V Lock. Spear-Rod Adjustment on New Gates

Spear Rods

When the lock gates at the King George V lock were installed originally, it was decided to incorporate a self-adjusting spring device at the top of the spear rod (*Figs 11 (a)*). This was intended to provide automatic adjustment for unevenness in the roller path or small obstructions which might be occasionally experienced in operating the gates. The weight of each

gate at the King George V lock is 350 tons,[2] but the coil spring, which was originally designed to give a total load of 4 tons (the minimum preponderance of weight over buoyancy on these gates being 5 per cent, that is 7·5 tons) was ineffective in practice. This was partly owing to lack of greasing, but had apparently been allowed for in the original design, which made it a comparatively simple matter to convert the spear-rod fixing to the more rigid wedge-and-cotter arrangement (*Figs 11* (*b*)).

Later, as a result of experience with these gates, it was decided to adopt a simpler form of adjustment on the pair of spare all-welded-steel gates which were installed at the King George V entrance in 1951. The usual guides were extended above the tank top level of the gate, reinforced by gusset brackets, and bridged over at the top by a plated strongback. A hydraulic jack was inserted between the top of the spear rod and the plated strongback, with the gates in the mitred position with small positive head of pressure against them. The spear rod was jacked down until the gates were level and the clearance between the strongback and the top of the spear rod was accurately measured. When two plate liners of appropriate dimensions had been bolted in position on either side of the jack, the latter was removed. Small adjustments can be made by inserting washers at the top of the spear rod as indicated in *Figs 11* (*c*).

Repairs to Roller Paths

Where gates are fitted with rollers, the roller paths have been inspected by diver and, when possible, closer examination has been carried out by means of a diving bell.[3]

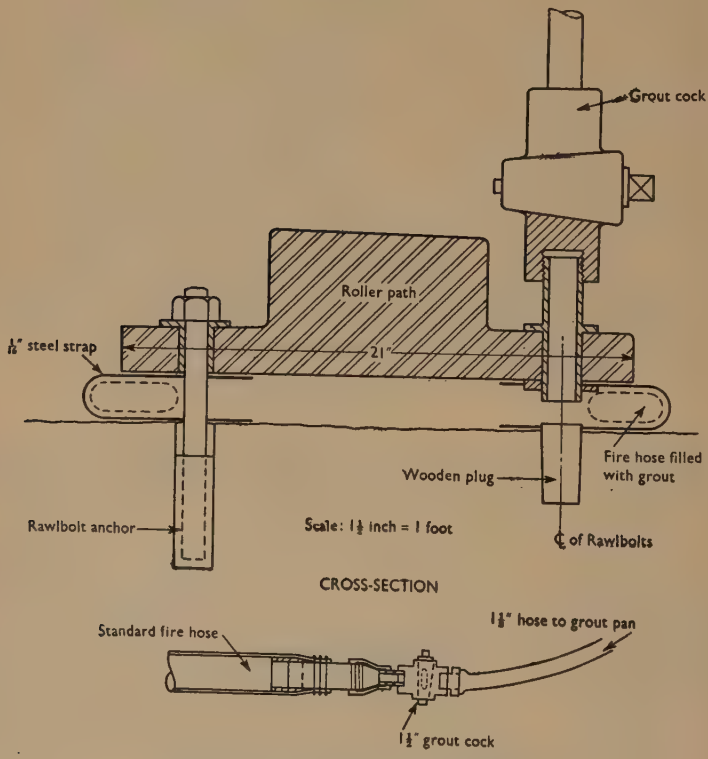
Some sections of roller path have been re-laid and it is interesting to record a method employed to lay and fix in position the sections of new roller path for the outer gates at the East India Dock (*Figs 12*). The sandstone apron had worn so irregularly that the new path could not be laid directly upon it, and it was therefore necessary to lay a new bed. To determine the correct path level, a length of $1\frac{1}{2}$-inch-diameter tubing with a 2-foot rule clamped to its upper end and supported vertically by four guy ropes, was positioned by the diver alongside each of the jack bolts supporting a section of roller path. The holes for the holding-down bolts were drilled using the new sections of path as templets. To contain the new bed of cement grout, $2\frac{3}{4}$-inch-diameter fire-hoses were laid underneath the flange of the path, to which a cock was fitted at each end, and $\frac{1}{2}$-inch-diameter pipes were laid to the surface. One end was connected to the grout pump, the other being left free. With the cocks open a thin grout of pure cement was pumped through, and when this was discharged

[2] Asa Binns, "The King George V Dock, London." Min. Proc. Instn Civ. Engrs, vol. 216, p. 372 (1927–23, Pt II).

[3] R. R. Liddell, "Improvements at the Royal Docks, Port of London Authority." Instn Civ. Engrs, vol. 10, p. 283 (Jan. 1939).

at the free end the hose cock at the corresponding end of the roller path was shut by the diver. Pressure was then increased to 100 lb. per square inch and the hose cock at the other end of the roller path was closed. After 2 days it was found that the grout-filled hoses formed an effective seal on each side of the path, and the end gaps between the hoses were

Figs 12



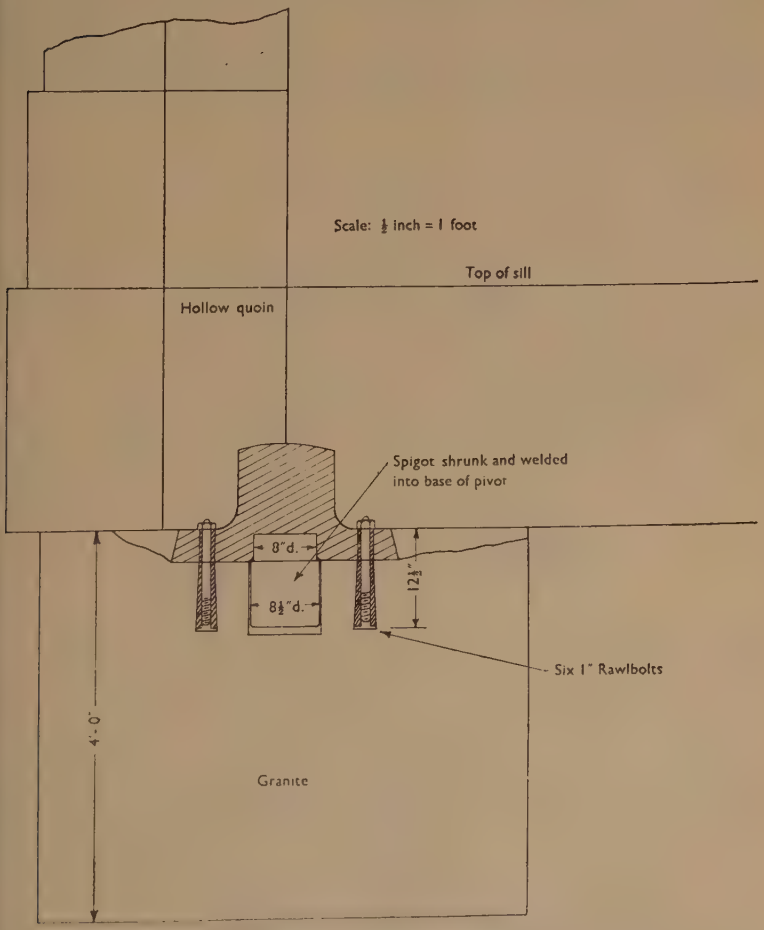
East India Docks. Repairs to Roller Path

plugged. A 1 : 1 sand/cement mixture from a colloidal concrete-mixer was then forced into the centre void by means of an 8-cubic-foot grout pan. The grout supply line was connected to various bolt-holes until all voids had been filled, and the whole was subjected to an air test on completion with satisfactory results.

Upper Outer Gate, South Dock Entrance, West India Dock

The upper outer gate of the South West India Dock entrance was damaged when a bomb, after striking the rolling-lift bridge, exploded in the lock. The original pivot was 12 inches in diameter and 9 inches high, with a 4-inch-thick base about 2 feet 4 inches in diameter. The base was sunk into a 4-inch recess in the top of the granite pivot stone, at the base of the hollow quoin. There were no holding-down bolts. In 1942, as a result of the above incident, approximately 4 inches at the top of the

Fig. 13 (a)

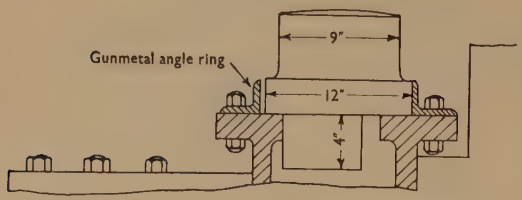


West India Docks. South Dock Entrance. Repair to Bottom Pivot

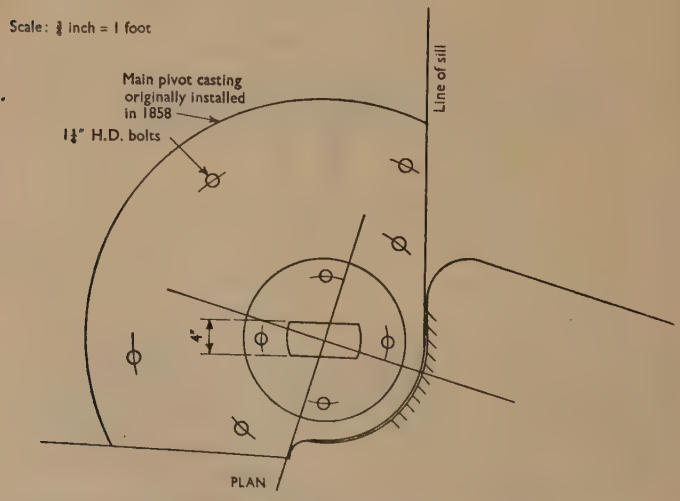
granite stone sheared away and the pivot was left free to move sideways. In an attempt to refix the pivot, six $1\frac{3}{4}$-inch-diameter holes were drilled in the flange and, through four of them, 1-inch cinch bolts were fitted and let 4 inches into the granite. In the remaining two holes, $1\frac{3}{8}$-inch-diameter wedged dowels fitted with nuts were sunk 8 inches into the pivot stone. The size of the stone precluded the enlarging of the flange by a suitable collar, because the concrete bearing face towards the lock was of doubtful value. The work was completed in little more than a week.

In 1949, a similar mishap occurred when the gate again carried away, shearing the pivot off its seat. This time, the old fixing bolts were abandoned and six new $1\frac{3}{4}$-inch-diameter holes were drilled in the granite on the same pitch-circle diameter as the old holes, which were grouted up. The new holes were drilled 7 inches deep and six dowels were fixed through the existing holes in the pivot flange. These were really intended to fix the pivot in a lateral direction and were provided with $2\frac{1}{2}$-inch-diameter collars to fit into recesses on the underside of the casting. The weight of

Figs 13 (b)



PART SECTION THROUGH MAIN CASTING SHOWING RENEWED PIVOT



SURREY DOCK. BASIN GATES. RENEWAL OF BOTTOM PIVOT

he gate bearing on these collars served to prevent the dowel bolts from sing.

In 1951, the bolts sheared again. In view of the obvious magnitude f the side thrust exerted by the gate on the pivot, it was considered that greater resistance to shear was required. A limpet dam was placed in osition and the existing holes were drilled out to a depth of 9 inches in ne granite and of suitable size to take 1-inch-diameter Rawlbolts of the ose-bolt type. On the centre-line of the pivot, a hole 9 inches in diameter nd 9 inches deep was sunk in the concrete. Into this was fitted the base f a spigot whose upper end was shrunk into the base of the pivot and nally welded round its circumference. The spigot was made of forged eel, and 1-inch clearance was allowed between the bottom of the base nd the granite (*Fig. 13* (*a*)) to ensure that the main casting was bedded tisfactorily.

Another interesting type of bottom pivot exists at the cuttings between ne Surrey Basin and the Island and Albion Docks of the Surrey Commercial Dock system. The base casting, which is fixed to the granite by olts, is provided with a slot which accommodates a spigot of rectangular ction on the underside of the pivot (*Figs 13* (*b*) and *14*). The pivots of ll four gates were found to be worn with at least 1 inch eccentricity, wing to excessive wear on the greenheart, and they were replaced by four ew pivots turned out of forged steel from a spare length of propeller haft (*Fig. 14*). The rollers of the basin gates were in a badly corroded ndition and had large flats where they had been sliding on the roller ath owing to the bearings seizing up. Since the gates weigh only 60 ns and are always swung with the water at the impounded level, it was cided to dispense with the rollers and spear rods. The preponderance weight over buoyancy of these gates has been adjusted to a value of tons at the impounded level, which is well within the capacity of the isting anchorages and pivots.

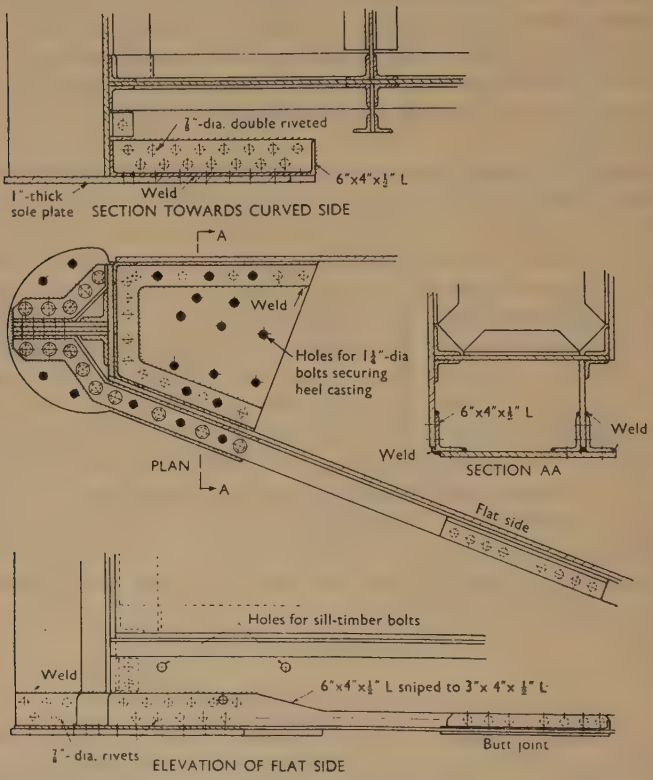
Modifications to Heel-Casting Fixings

The bottom gudgeon casting is usually bolted to a mild-steel plate out 1 inch thick which is attached to the gate structure by riveted gles. In 1949, it was discovered that these riveted connexions had iled on an inner gate at the King George V entrance lock, so that the sting was virtually detached from the gate, which weighed about 350 ns (*Figs 15*). The thickness of the shell plate had been so reduced by e abrasive action of the casting moving in its seat that it could not be ied upon for a new fixing. The three thicknesses of blade plate were so badly buckled at the bottom of the heel.

In repairing this damage the shell plates on both sides of the gate re cropped and a new section of plate inserted by butt welding. The ttom angles were increased to 6-inch-by-4-inch-by-$\frac{1}{4}$-inch unequal angles

to provide room for a double row of staggered-pitch rivets for connecting the angle frame to the shell plating of the gate, which was kept $\frac{1}{4}$ inch short for the length of the new section. The spaces between the bottom edges of the renewed parts of shell plate and the base-plate were welded solid. The back plate was built up by welding and the doubling plates and angles of the blade plates were cropped in staggered fashion and

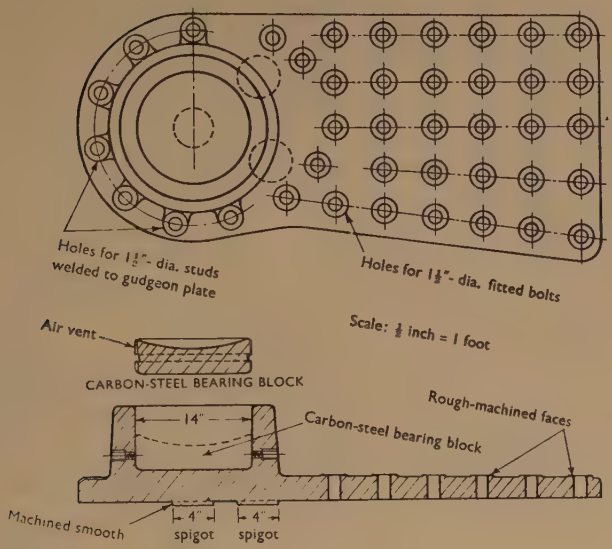
Figs 15



King George V Lock Gates. Sole Plate for Bottom Gudgeon Casting

part renewed. This failure was partly attributable to the heel-posts wearing beyond the 1 inch clearance provided on the pintle. The actual wear on the timber was $1\frac{1}{4}$ inch, so that the blade plate projected through the worn post. On the large new gates, a solid fixing has been provided for the bottom casting, which has three 4-inch-diameter spigots in its upper face to assist in taking the shear at this point (*Figs 16*).

Figs 16



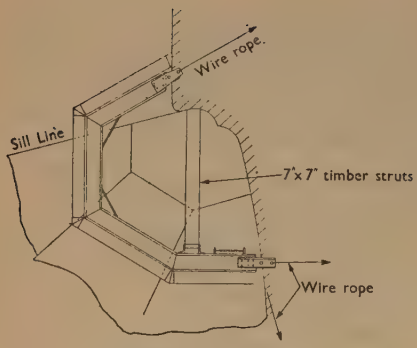
King George V Dock. Cast-Steel Bottom Gudgeon

Limpet Dams

The limpet dam referred to previously was constructed at Tilbury and ·sed for examination purposes ; but it was found to be too small for men • work in comfortably and the possibility of constructing a larger dam hich could be adapted for use at the majority of gate positions in the ocks was therefore investigated. Owing to the variety of the lock ctions and the probable necessity for having to carry out operations quiring the limpet simultaneously at more than one site, two other npets were eventually built, one of welded steel construction at the oyal group of docks and one of composite steel-and-timber construction Surrey Docks (*Figs 17*). The Surrey Docks limpet was used on a number occasions and it was eventually found possible to place it in position nd pump it dry within the comparatively short time of an hour and a alf.

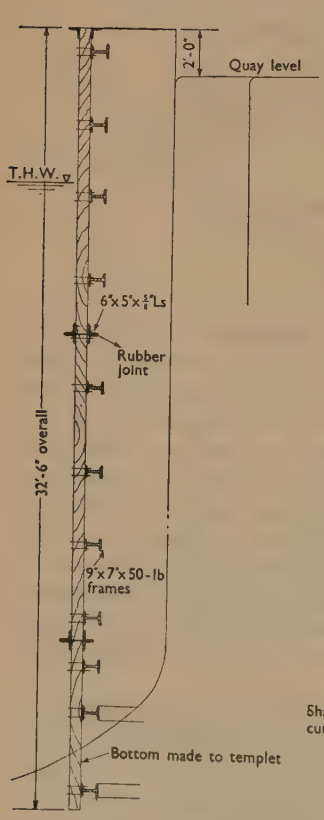
This limpet was built of 12-inch-by-6-inch timber bolted to specially bricated and welded steel joists with a bottom detachable portion which ould be changed to suit the particular shape and hand of the quoin being orked on ; the main section was reversed as necessary. The seal between e limpet and brickwork or masonry of the lock wall was formed by a nvas bag packed with oakum. The dam weighed approximately 12 ns in air and measured 32 feet 6 inches overall. It was loaded with

Figs 17 (a)

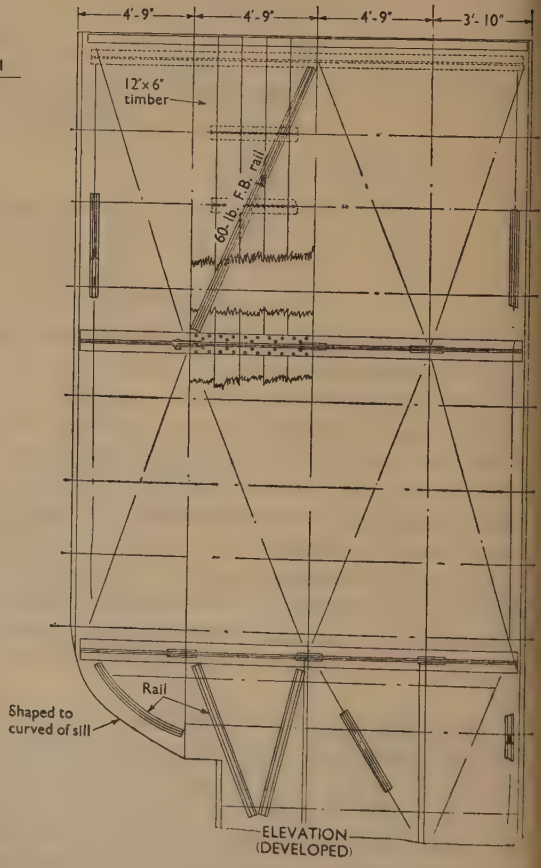


PLAN

Scale: $\frac{1}{8}$"=1 foot approximately



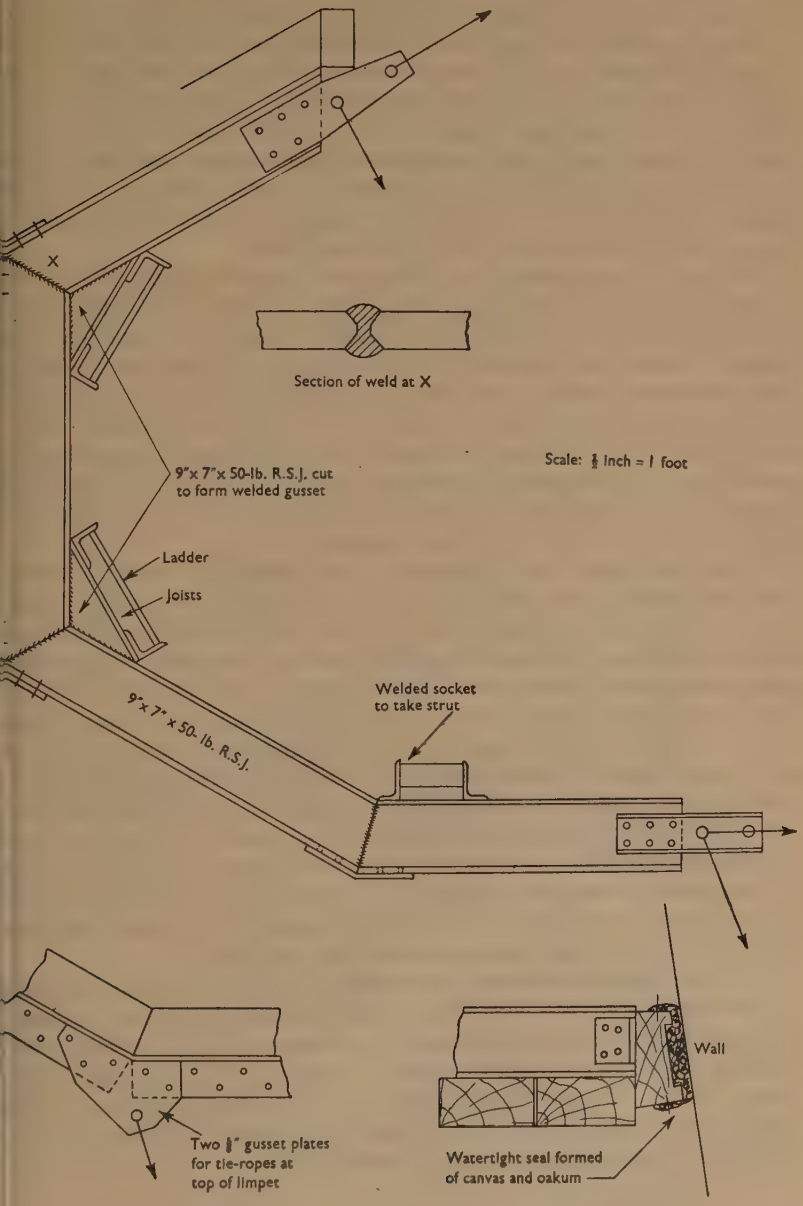
PART SECTION



ELEVATION (DEVELOPED)

General Arrangement of Typical Steel-and-Timber Limpet Dam

Figs 17 (b)


X
Section of weld at X
Scale: ½ inch = 1 foot
9"x 7"x 50-lb. R.S.J. cut to form welded gusset
Ladder
Joists
Welded socket to take strut
9"x 7"x 50- lb. R.S.J.
Two ⅝" gusset plates for tie-ropes at top of limpet
Wall
Watertight seal formed of canvas and oakum

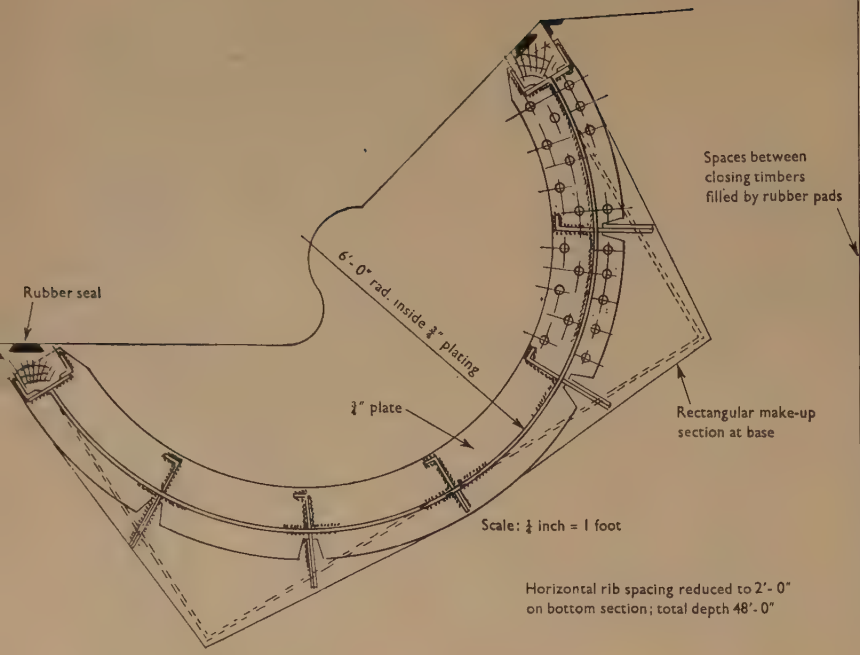
STEEL-AND-TIMBER LIMPET DAM. DETAILS OF TYPICAL FRAME

concrete kentledge blocks, when in position, to ensure a good seal at the bottom. It provided a working area approximately 6 feet square, which was found to be quite adequate. When the limpet was placed in position by a floating crane, a diver was in attendance to seal off any large gaps caused by irregularities in the lock walls by wedging oakum bags into the openings. As soon as the dam was reasonably fitted and pulled into position by the mooring wires, the suctions of three 3-inch-diameter pumps and one 4-inch pump were placed inside the enclosed area. This number of pumps was found by experience to be very effective in causing sufficient difference in head to press the limpet to the wall and complete the seal, the effectiveness of which is seen in *Fig. 18*.

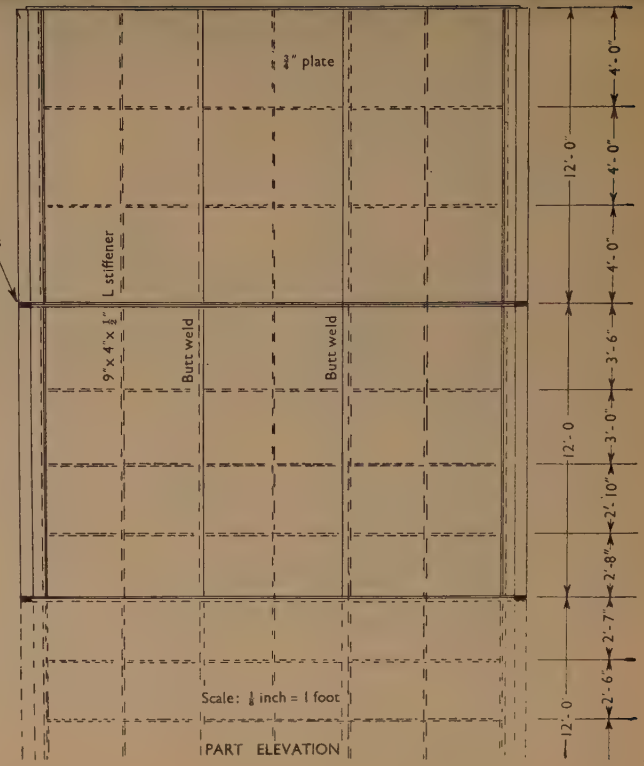
The cost of constructing this limpet was approximately £1,100, and the cost of handling, pumping, lighting, etc. amounted to about £80 per week. Once in position and sealed, it could be kept dry by one hydraulic ejector of about 1,000 gallons per hour capacity. In practice it appears that the friction between the limpet and the dock wall is sufficient to counteract the effect of any unbalanced force resulting from the hydrostatic pressure, but horizontal struts into the hollow quoin were placed in position as a precautionary measure. Rails (60-lb. F.B.) were also attached to the exterior to increase the preponderance of weight over buoyancy, and to act as additional bracing.

While the limpet was in position at the cutting between the Surrey Basin and Island docks, the opportunity was taken to repair the hollow quoin, which had been deeply scored by the heel-casting and rivet heads nearest the heel-post through heavy wear on the greenheart. The original quoin stones were of sandstone. The damaged portions were cut out to a depth of 4 inches, with dovetailed sides, phosphor-bronze dowels were grouted into the back of the stone and laced with steel wire, and the cavity made good by filling with granite concrete in 3-foot lifts behind a steel shutter formed to the appropriate radius. The proportions used were one part by weight of ordinary Portland cement to one part each of $\frac{1}{8}$-inch granite chips and granite dust, which gave a crushing strength of considerably more than 4,000 lb. per square inch at 14 days. The bottom of the quoin was treated in this manner for a height of nearly 9 feet, test cubes being made from each lift (*Figs 20*).

The steel limpet which was constructed in connexion with the overhaul of the various lock entrances at the Royal group of docks was, in view of the greater depth of water, designed as a portion of an all-welded steel cylinder (*Figs 19*), and will be used for repairing the hollow quoins and bottom pivots. In some cases 45 feet of water is available over the sill, and a timber dam, which would require additional ballasting for this depth, would probably prove very cumbersome.


Rubber seal
6'-0" rad. inside ¾" plating
¾" plate
Spaces between closing timbers filled by rubber pads
Rectangular make-up section at base
Scale: ¼ inch = 1 foot
Horizontal rib spacing reduced to 2'-0" on bottom section; total depth 48'-0"

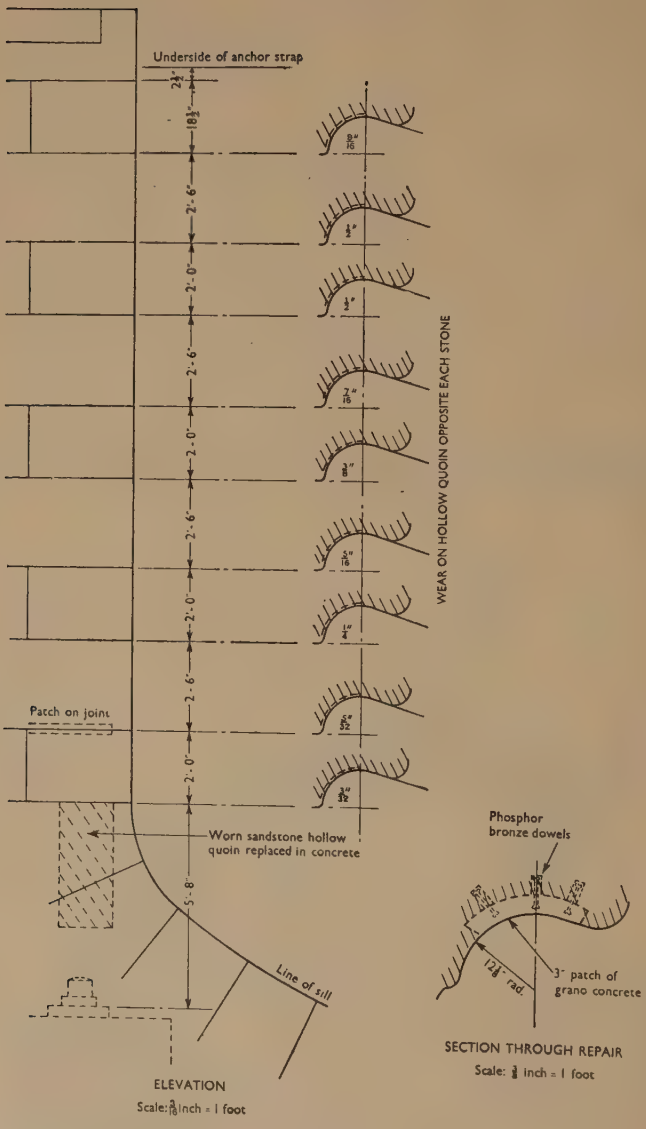
PLAN VIEW AT BOTTOM SECTION



PART ELEVATION

ALL-WELDED MILD-STEEL LIMPET DAM

Figs 20

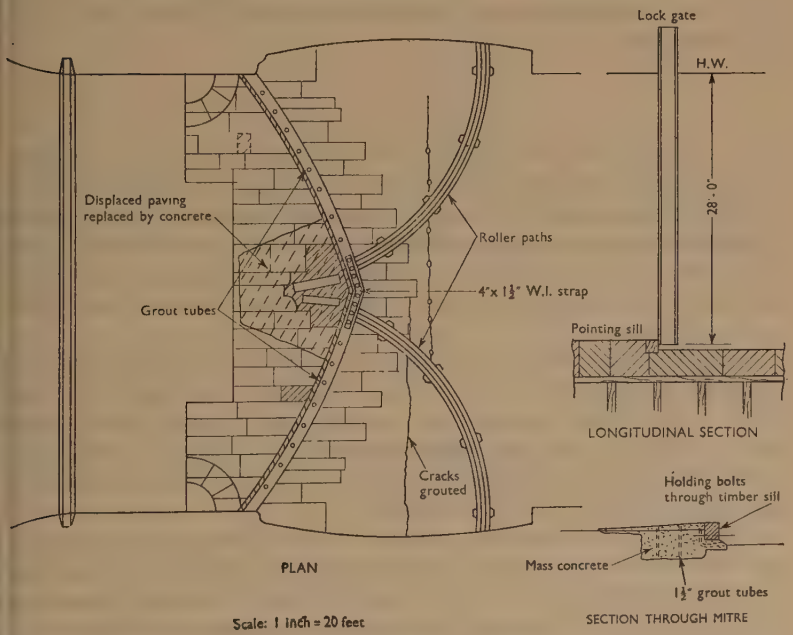


SURREY DOCKS BASIN. REPAIRS TO HOLLOW QUOIN

Underwater Repairs to Pointing Sill

The entrance to the St Katharine Dock, situated on the north bank about ½ mile below Tower Bridge, is provided with a timber-faced pointing sill. A diver's survey of this particular sill showed that part of the masonry in the sill platform was missing completely and that there was an uninterrupted passageway for water below the sill timbers, which had been forced out of their recess. In addition, the steel-plate cover strap holding the sill timbers together at the apex had become detached (*Figs 21*).

Figs 21



St Katharine Dock Entrance. Underwater Repairs to Pointing Sill

The Authority's diving bell was used for this repair, after certain modifications including the separation of the low-pressure and high-pressure air intakes. A new gland was made and a separate high-pressure supply for drills brought in at a lower level than the low-pressure air. The bell, weighing 16 tons in air, was handled by an 18-ton-capacity crane on a special vessel hired from a contractor.

A thorough inspection from the diving bell confirmed the helmet diver's report that the sill timber itself was actually in good condition. The steel cover-strap was re-drilled and fixed over the joint at the apex of the timber sill face after the timbers had been forced back into position by the weight

of the bell. Additional bolts were let into the timber both horizontally and vertically and concreted in. The concrete was mixed in a 5–3½ machine placed on the deck of the crane vessel, and ¼-yard-capacity bottom-opening skips were taken down at each lowering of the bell. The holes at the back of the timbers and at the face were filled with 1½ : 1½ : 3½ concrete, using rapid-hardening cement. Grout tubes were left in the concrete and fixed through the timbers, and neat cement grout (equal parts cement and water) was pumped in to seal all cracks and joints. All the grout tubes were cut off level with the floor and any cracks in the sill were caulked-up and pressure-grouted by helmet diver. The whole of this work was carried out in 1 month by a diver shipwright working in the bell, assisted by ten men including crew, a helmet diver and attendant, and an electrician.

Dimensions of Gates

During the 1939–45 war, many of the engineering records were destroyed by enemy action, including some of those showing the main dimensions of the lock entrances and the records of previous repairs to the lock gates. Such original drawings as were available could not always be relied upon for this information, and therefore it was necessary to re-establish these dimensions by fixing the centre lines of hollow quoins and angles of pointing sills on the site.

At the Tilbury lower-entrance lock the angle and rise of the sill were checked by a diver using a templet constructed from two pieces of ¼-inch mild-steel plate bolted together at the mitre end through slotted holes. The length of each plate was approximately the length of a gate, and a hole was cut at one end corresponding to the bottom pivot diameter so that the plate could be fixed at this end.

A different type of templet constructed from timber, for floating on the surface or lowering under water in a weighted condition, was developed for ascertaining the exact finished length of gates at Surrey Commercial Docks (see *Fig. 22*). The first leaf of a pair of gates was trimmed to a length indicated from the diver's measurements under water using the weighted templet and checked against available drawings. After this leaf had been stepped it was brought to the pointing sill and the timber templet was then floated into the position to be occupied by the second leaf with one end in the hollow quoin, and a pointed adjustable feeler at the mitre end was adjusted by butterfly screws so that it was just held in position by the joint in the mitre timbers of the first leaf. By this means the actual length of the second gate required for a good fit at the top of the gates was obtained. A similar process was carried out for the bottom of the gates, the templet being weighted and placed over the bottom pivot. Correct levelling was ensured by resting the mitre end of the templet on a small batten which had been temporarily nailed at a predetermined level

to the mitre timber of the first gate. By this means, the gates were made to fit accurately in spite of any local changes in the main dimensions which might have occurred through small movements in lock walls or settlement from some other causes. Despite the fact that most of this work had to be carried out by diver working almost completely by touch owing to the quantity of silt in suspension in the dock-water, very satisfactory results were obtained.

Heel-Posts, Mitres, and Sills

This heavy programme of gate repairs had to be commenced when there was some difficulty in obtaining greenheart logs of the necessary dimensions and quality. Whereas the granite quoins may only require attention once in 50 years—if then—it usually happens that a gate requires the renewal of both the heel- and mitre-post, which in the case of large gates means logs of considerable size. In 1949, the import of greenheart from British Guiana was restricted, there being only two vessels engaged in the trade. Altogether nearly 10,000 cubic feet of greenheart, in two hundred logs of various lengths, was required over a period of 18 months and, with the co-operation of the suppliers, the timber was eventually included in shipments to a prearranged delivery schedule. Where possible, the timber was sawn to convenient dimensions before inspecting ; but this was not always possible and a thorough inspection was then required, as cases have occurred of worm-affected timber not coming to light until actually fixed on the gate and being finally dressed. Some gates have curved clapping sills in addition to circular heels, which results in much shaping of the log, so that the finished cost of the greenheart work is comparatively high. The cost of the material actually used ranged from £2 to £4 per cubic foot, and the cost of labour involved in shaping and final dressing may be as much as £4 per cubic foot. The total cost of the finished timber fixed to a gate, including allowance for waste, ranges from £10 to £12 per cubic foot. The variation arises from the difference in the amount of timber wasted and the apparently higher cost of dressing a flat surface as compared with a curved surface. A contributory factor in this high cost is the tolerance which has to be allowed on the finished dimensions for dressing. On some drawings of new gates as much as 3 inches has been shown. This, it is considered, could be reduced with advantage.

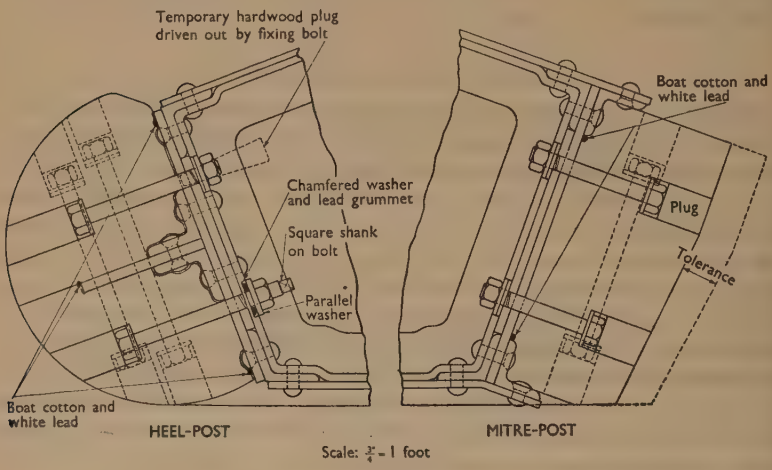
For inspecting the logs at the merchant's yard it was found useful to apply a templet of the finished shape of the heel, sill, or mitre concerned in order to ascertain whether the log could be used economically and to the best advantage. For example, the heart of the timber should be placed in the bosom of the fixing angles and the annular rings on the outside of any radius. For this purpose it is advantageous to select logs with the heart out of centre if possible, thereby reducing cutting.

To ensure complete watertightness of the gate, careful attention was paid to the fixing of the greenheart to the steelwork, because, under the

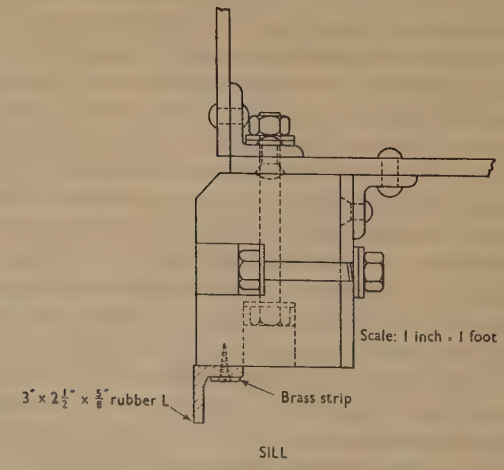
impacts generally met with in practice, there is a tendency for the fixing bolts to work loose. A modified form of fixing bolt, as shown in *Figs 23* (*a*), has been used, incorporating a lead grummet which is tightened

Figs 23

(*a*)



(*b*)



TYPICAL GREENHEART FIXINGS

by the action of a parallel and chamfered washer. The end of the bolt inside the gate has a squared shank which permits subsequent tightening from inside the gate chamber. When a greenheart heel is offered up to

the gate the really effective bearing surface is limited to the flat areas between the rivets over which the timber is recessed, and a tight fit is very difficult to obtain. The timber is usually bedded in red lead paste, but it has been found that under pressure water will leak between the timber and steel on heels or mitres. To prevent this, a "stop-water" made from boat cotton and white lead is fitted in the position shown, and when the timber is tightened against this an effective seal is formed (*Figs 23* (*a*)). In some cases where granite pointing sills were found to be slightly damaged, the fitting of $\frac{5}{8}$-inch rubber angle to the bottom leading edge of the clapping sill has been found helpful in reducing the leakage (*Fig. 23* (*b*)). On the whole, this has been found to be a success, but it is vulnerable where underwater obstructions are experienced, and if for some reason the gates are not correctly mitred, the scour set up by the difference in head forcing the water through the small gap between the gate and sill causes the rubber to vibrate violently and probably tends to weaken the fixing.

A further method of ensuring a really good fit by the fixing bolts has been adopted by first driving a hardwood plug into the hole through the steelwork. White-lead putty is then inserted into the hole in the timber for a depth of about 1 inch, and a square-headed galvanized bolt is then driven gradually into the hole to force the white lead into all interstices between the bolt and timber. The bolt is finally driven hard to eject the wooden plug (see *Figs 23* (*a*)).

All dock engineers are familiar with the methods used in shaping and dressing the timberwork on gates, and it is not intended to describe this part of the work in detail. Generally the timber was adzed by hand to rough shape and dimension, and, using a piano wire and distance block, finally planed to the required net dimensions. Centre marks were made on heel-castings and platework, and lines were scribed on the platework to show a known dimension from the mitre end for future reference. The stripping, fitting, and rough dressing were carried out at negotiated piecework prices and the final finishing at daywork rates.

The mitre-posts for direct-ram-operated gates were usually finished with a 1-inch to $1\frac{1}{2}$-inch chamfer on the vertical edges; but for chain-operated gates, which do not mitre so easily, the chamfers were increased a little to allow more latitude in bringing the gates together.

Gate Platforms

Experience appears to show that timber is the best material for the gate platforms. The main kerb timber on the flat side of the gate should be about 12 inches by 6 inches and the timber platform of 9-inch-by-3-inch or 12-inch-by-4-inch decking. In some cases, the outer lock gates are within a few feet of the entrance jetty and it is quite usual for them to receive a glancing blow from a vessel. The timber platform and fendering

absorb an appreciable shock by distributing the load and partly transmitting it to the lock wall. They are therefore preferable to the steel-framed platform which is really only suitable for a middle gate position and is more vulnerable to damage.

Timber Fendering on Gates

Both vertical and horizontal timber fendering have been used on the non-pressure side of the gates. In addition to the impact from ships, the heavy barge traffic of the Port of London imposes a great strain on such fendering, and in the Author's opinion the horizontal method of fixing as employed at the Greenland entrance gates is the most satisfactory; 12-inch-by-6-inch elm has usually been employed for this purpose. Vertical walings are fitted to the gate by angle brackets and the horizontal timbers are fixed to these by galvanized coach screws instead of spikes, which work loose under impact. A 14-inch-square timber stop about 6 feet long, bolted to the back of the gate recess in line with a vertical diaphragm in the gate, has been found adequate to support the gate against heavy blows. If the stop is carried more than about 6 feet below the cope level, there is a tendency for flotsam, particularly timber, to collect behind the gate and to prevent it opening to the full extent.

Gate Machinery

At the present time, all the gate machines at the Authority's docks are worked by hydraulic power at about 800 lb. per square inch nominal pressure, but electrically-driven machines are being incorporated in the reconstruction of the lower Gallions entrance lock. There are, however, certain advantages in retaining hydraulic power for gate machines when it is readily available. There is probably less chance of breakdown, but this is no advantage at the Gallions entrance where the machines producing the hydraulic power are powered by electricity in the first instance. With hydraulic power there is less strain on the gate machinery when it is necessary to keep the gates "pointed" or partly closed against a "run in," when the river level exceeds the impounded level.

Chain-operated gates have given satisfactory service for long periods, but, in addition to the limitations which they impose on the effective depth of the lock, there is the disadvantage that the chains are vulnerable to damage. One of the main causes of damage is the trapping of the chain between the gate and the sill. Since this usually breaks the chain near the gate end, it has been found an advantage to fit a pennant about 15 feet long at the gate end to save the renewal of a complete chain.

The design of the chain attachment to the gate is important. A strong cleated bracket riveted to the gate skin with an eyehole for the shackle pin is one of the most satisfactory, and it is advisable to provide

this in duplicate. During the course of repairs any wastage in this bracket was made up by welding. If any slack rivets were discovered, the entire bracket assembly was dismantled and the holes reamed out prior to re-riveting.

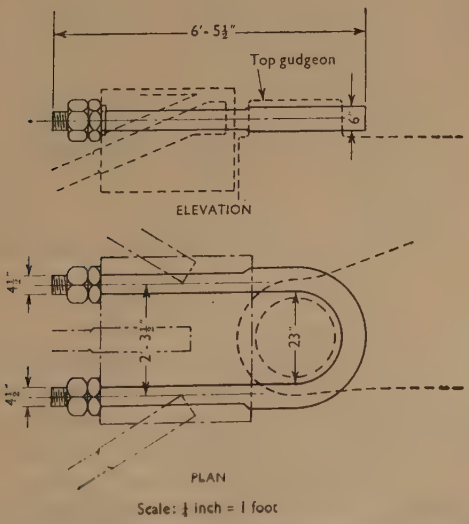
Crocodile Girders

When a gate is operated by a ram connected to a crocodile beam having an independent anchorage fitted on top of the gate, there are certain advantages gained from the absorption of the violent forces which sometimes occur. There are also disadvantages associated with this type of fixing, for the gate anchorage cannot be reached until the gate platform and supporting members have been dismantled and the crocodile has been removed. It is usual also for the access trunks to buoyancy chambers to project through the crocodile beam, which has therefore to be lifted off vertically by a floating crane. In some cases, therefore, the top portion of the access trunks has been rebuilt as a separate section, which facilitates movement of the crocodile beam.

Anchorages

As is to be expected among a large number of gates of varying age and size, several types of anchorages exist. Generally, the most satisfactory from the operating and maintenance aspects is the plain U-strap fitted with nuts for easy adjustment (*Figs 24*). With this type of strap the

Figs 24



Typical U-Strap for Top Anchorage of 350-ton Gate

operation of stepping a gate is much simplified, and adjustments for wear and other causes are fairly easily carried out. It has been found that, in practice, the U-strap fitted with wedges and cotters gives only very limited adjustment and in some cases none at all.

In fitting the screwed U-strap type of anchorage, care should be taken to thread the thin nut on first so that the load is taken by the thread of the large nut. If the nuts are put on the other way round, the probability is that after a few years they will have to be burnt off.

The later form of drop-over plate and link anchorage is not considered to be so satisfactory. This type of anchorage gives practically no support against thrust, consequently the gate heel-post tends to wear, and, unless the anchorage is kept constantly in good adjustment, there is a risk of causing serious damage.

Any appreciable wear on the top gudgeon through contact with the anchorage strap was built up by welding and machined to avoid damage to the heel-post, which might be caused by the eccentricity bringing the timber into contact with the leading edge of the quoin stones.

Casting Repairs

An interesting repair was carried out to the cracked top-anchor-block casting of the inner upper gate at the Greenland entrance to Surrey Docks. It was known that the gate had been severely strained during the 1939–45 war, but the full extent of the damage was not discovered until the removal of the crocodile-girder anchor-plate revealed a fracture for the full depth of the main anchor casting, to which the gate U-strap is attached. Using a patent process developed during the war, a series of holes was drilled across the fracture, to a depth varying with the thickness of the parent metal. Special-alloy keys were worked into the holes by a pneumatic tool to cold-weld the two portions of the casting together. The vertical length of the fracture, where the casting formed the top of the hollow quoin, was actually repaired in a little more than 2 hours between removing the gate and stepping its replacement by floating crane. The whole repair was completed in 7 days without interruption to the use of the lock, the gate being worked temporarily by capstan and wire (see *Fig. 25*).

Corrosion and Painting

The structures of most gates were remarkably free from corrosion, apart from the top strakes of thinner shell plating between wind and water, and the tank tops, which are usually made inaccessible for periodical painting by the platform and crocodile girders. Corrosion was also sometimes apparent in the shell plating adjacent to the timber walings for the fendering. Some of the gates have been repainted internally with red lead, but in view of its high cost and doubtful value on badly pitted surfaces it has been more usual to apply two or three coats of

bituminous compound internally and externally. Immediately after scaling, a coat of bituminous solution was applied with the plating in a perfectly dry and clean condition, and, on completion of all exterior work, the gate was given a final coat of bituminous enamel, which, when applied correctly, remains fairly soft on drying and does not become hard and brittle. The various mixtures of heated coal-tar and of Russian tallow and unslaked lime, applied hot, had evidently been used on many of the gates handled, with excellent results, and a mixture of 9 gallons of coal-tar to 2·25 quarts of naphtha and 13 lb. of slaked lime, was also used in the current programme.

When applying thick coatings of bituminous enamel to ballast chambers, it is important to ensure that any small vent holes in the top deck are left free. If these holes are blocked there is a possibility that air will be trapped at the top of the ballast chamber, thus reducing the preponderance of weight over buoyancy, which may become critical when abnormally high tides are experienced. This difficulty can be overcome by increasing the diameter of the vent holes to about 3 inches, or by fitting the angle-bar reinforcing surrounds to the manholes on the upper side of the top deck of the ballast chamber instead of the lower side. The larger-diameter holes would also facilitate the handling of heated rivets during repair work.

Costs and Methods

The major part of the repairs described or referred to in this Paper was carried out by the Port Authority's appointed maintenance contractors on mechanical and marine works. This meant that, although it was found possible to arrange piecework rates and lump-sum amounts for certain operations—such as scaling, painting, and stripping and fixing timber—the greater part of the work was done on a time-and-lime basis, and difficulty was experienced, particularly during the early part of the programme, in making as much progress as desired. The possibility of drawing up a priced schedule, which it was thought would assist in expediting the work, was therefore examined, but so far it has not been possible to introduce such a schedule for the gates repaired in this programme, owing mainly to the lack of reliable information about the exact condition of the gates, following the interruption in periodical repairs. Under normal conditions in the future, when a regular routine of maintenance has been re-established, there appears to be no reason why the repairs should not be carried out on a schedule basis as they were prior to 1939. There could be, for example, in the schedule, one price for cleaning and scaling the gates with extra over rates for special processes such as mechanical chipping. Plates could be measured by weight according to the plate thickness, rivets and bolts by number, angles by weight and dimensions, make-up welding by volume, etc. In the case of timber the material

could first be paid for in bulk and the work of fitting and dressing charged at a price per cubic foot. Lump sum prices could also be negotiated for painting, the overhaul of ejectors, heel-castings, roller carriages, and sluice gear.

Costs of Gates and Repairs

From Table 3 it will be seen that the cost of completely overhauling a pair of gates for an entrance 80 feet wide and 35 feet deep, or 100 feet wide and 45 feet deep, was equivalent to £9 5*s* per square foot of waterway area, the total cost of £25,000 being roughly divided between the various trades as follows (this cost being between three and four times that obtaining in 1936) :

Handling, floating craft, etc.	10 per cent
Stripping for repair	5 ,,
Cleaning and scaling	15 ,,
Repainting	7·5 ,,
Structural repairs	25 ,,
Sluice gear, rollers, etc.	12·5 ,,
Timber work (including fendering)	25 ,,

In 1912, the cost of a pair of steel lock-gates for an entrance 80 feet wide and 40 feet 6 inches deep over sill at high water, ordinary spring tides, amounted to about £9,500, which was equivalent to about £2 10*s* 3*d* per square foot of waterway area. Gates at Liverpool for somewhat wider entrances cost £3 10*s* per square foot during the same period.[4] The equivalent present-day cost for all-welded steel gates appears to range from £15 per square foot for smaller entrances to £10 per square foot for larger entrances, depending upon the type of anchorage and operating machinery employed.

The comparatively high cost of repairs was considered to be worth while in view of the long delivery required for new gates, which take 18 months to 2 years to complete. However, the Author believes that, no doubt because of the prevailing conditions in the industry, the cost of repairs in London is about double that in the provinces. It would be interesting to know if this view is confirmed by experience at other ports.

Acknowledgements

The information in the Paper has been made available by the kindness of the Chief Engineer of the Port of London Authority, Mr G. A. Wilson, M.Eng., M.I.C.E., M.I.Mech.E.

[4] T. L. Norfolk, " Notes on the Design and Construction of Modern Dock Gates." *Trans. Liverpool Engng Soc.*, vol. 38 (1917), p. 49.

The Author also wishes to express his grateful appreciation to Messrs Harland and Wolff, Ltd, Head Wrightson & Co. Ltd, Metalock (Britain) Ltd, and all colleagues on the Port of London Authority staff who have assisted with information.

The Paper is accompanied by eight photographs and fourteen sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared, and by the Tables printed in the following Appendix.

APPENDIX

TABLE 2.—PORT OF LONDON AUTHORITY LOCK GATES

Position	Pairs	Construction	Machinery
London Dock			
Shadwell entrance	3	Timber	Hydraulic chain
St Katharine entrance	2	Timber	Hand chain
Garnet Street lock	3	Timber	Hydraulic chain
Wapping entrance	2	Timber	Hand chain
Wapping cutting (not in use)	1	Timber	Hand chain
Surrey Commercial Docks			
Greenland entrance	3	Double-skin iron	Hydraulic ram
Greenland-S. dock	1	Timber	Hydraulic chain
Surrey entrance	1	Double-skin steel	" "
	1	Double-skin iron	" "
Surrey Basin	2	" "	" "
Greenland-Russia dock	1	" "	" "
South dock (not in use)	1	Timber	" "
Canal lock	3	Single-skin steel	Hydraulic ram
India and Millwall Docks			
South dock entrance	3	Double-skin steel	Hydraulic ram
India and Millwall Docks			
Blackwall entrance	3	Double-skin steel	Hydraulic ram
Millwall entrance (not in use)	3	Double-skin steel	Hydraulic chain
East India entrance	2	Double-skin steel	Hydraulic chain
East India inner lock	2	Double-skin steel	Hydraulic ram
Royal Docks			
King George V entrance	4 (One pair steel welded)	Double-skin steel	Hydraulic ram
Royal Albert dock			
Upper Gallions lock	3	Single-skin iron	Hydraulic chain
Lower Gallions lock	3	" "	" "
Royal Victoria dock entrance	3 (One pair steel welded)	Double-skin iron	Hydraulic chain
Tilbury Docks			
Lower entrance	3	Double-skin steel	Hydraulic chain
Upper entrance	3	" "	Hydraulic ram

TABLE 3.—SCHEDULE OF COMPARATIVE COSTS

	Principal dimensions of entrance	Gate material	Weight of gate : tons	Date of construction (gates)	Original cost per sq. ft. of entrance (incl. machinery) : £	Time in service since last repair: years	Cost of overhaul per pair of gates: £	Cost of repairs per sq. ft. of entrance : £	Estimated cost of new steel gates per sq. ft. of entrance (excl. machinery) : £	Remarks
London Dock										
Shadwell entrance	60′ by 28′	Timber	75	1855	1·15	24			12·00	Existing timber gates being replaced by all-welded steel gates
St Katharine Dock	45′ by 28′	,,	45	1828	1·72	26			15·00	
Surrey Docks										
Greenland entrance	80′ by 35′ 3″	Iron	165	1904	1·86	20	25,000	9·25		
Surrey Basin	50′ by 25′	,,	55	1858	1·65	40	8,500	6·70		
Surrey lock	50′ by 27′ 3″	,,	60	1858	1·54	16	17,000	13·00		Including underwater repairs
West India Dock										
S. dock entrance	80′ by 35′	Steel	160	1929		22	16,000	5·70		
King George V Dock	100′ by 45′	,,	350	1921	10·8	29	41,700	9·25	10·16	Spare pair of all-welded steel gates supplied.
Royal Albert Dock										
Gallions upper entrance	80′ by 30′	,,	100	1879		16	18,500	8·00	10·60	New gates under construction.
Gallions lower entrance	80′ by 36′	,,	142	1886		30			14·00	New all-welded steel gates to be supplied under entrance reconstruction scheme.
Tilbury Docks										
Lower entrance lock	79′ 6″ by 44′	Steel	175	1886	2·67	13	29,000	8·2		
Upper entrance lock	110′ by 45′ 6″	,,	330	1929	7·50	23	30,000	6·05		
Royal Victoria Dock	80′ by 20′ 6″	Iron	80	1857		22			14·96	New gates under construction.

Discussion

The Chairman, in proposing a vote of thanks to the Author, said that those responsible for the maintenance of dock gates realized that that was not just another branch of engineering but was, in many ways, a science of its own—especially in view of the costs involved. As an example, the contract figure for a pair of gates built in 1945 had been £38,759, and the cost of the site-work £15,761, making a total of £54,520. A current contract, let within the past few months, for a pair of gates of similar size, showed a big difference. The contract price for the gates was £66,000, and, after allowing a similar figure for the site-work, the total cost would be about £82,000. That was a great deal of money for what to the lay mind were two simple structures. He thought that all those concerned with the subject would agree with him that the subject of dock gates was not only an interesting one but one of vital importance.

Mr J. H. Jellett applauded the Author's unequivocal statement, in the first paragraph of the Paper, that " Under normal circumstances, it is considered advisable to overhaul lock gates at intervals of not longer than 15 years." Members of the Division might perhaps recall that, about 5 years previously, that matter had been discussed in connexion with a Paper which he had presented.[5] On that occasion, if he remembered rightly, the fact that the gates of the Port of London Authority had remained in position undisturbed for from 20 to 27 years had been referred to as a sign of exceptional virtue. He was very glad to note that apparently judging by the coincidence of the dates, the discussion on that Paper had given rise to second thoughts.

He was interested in the Author's remark that the docking of gates in the vertical position had been found to prolong the operations of repair. Was that attributable entirely to the position of the gates in the dock or could it be ascribed also to the additional factor to which the Author referred, namely, that the dock had at the same time been used for ordinary ship-repairing purposes, which had involved periodic floodings with the gate in position ? Mr Jellett asked that question because he would have thought that, assuming the exclusive use of the dock, the gate, once placed in position upright, might be considered, by reason of the large reduction of overhead work compared with that required with the gate docked in the flat position, to be in a more convenient position for working.

It was proposed next spring to remove for docking and maintenance purposes the caisson from the King George V Graving Dock at Southampton, which had not been removed for 20 years, and, owing to its dimensions and to the dimensions of the only graving dock which could be

[5] J. H. Jellett, "Maintenance Operations 1946–47 on the Box Gates at Dover Train-Ferry Dock." Maritime Paper No. 11, Instn Civ. Engrs, 1949.

made available for the purpose without seriously disturbing the normal docking programme, it would be necessary to turn the caisson over on its flat. That was regarded not as an advantage, but as a serious operation forced upon them by circumstances ; Mr Jellett had thus been interested to read the Author's opinion that he preferred gates (with which he would, it was to be hoped, classify caissons) turned on their flats for docking purposes.

The Author used an expression on p. 49 which Mr Jellett felt sure was intended to read slightly differently from the way in which it had been worded, namely, " since the period covering one tide was usually required to carry out any of the above operations." In Mr Jellett's experience, it was not that all that period was usually required to carry out the operations, but that they had to be completed within that time, lest the next low tide found the gate, caisson, or whatever it might be in some awkward position where it was likely to strand or suffer some other form of marine disaster. Perhaps the Author would make clear what were the circumstances in the cases under his supervision.

Reference was made in the Paper to the wear of the mitre-posts, and Mr Jellett thought, therefore, that it might be of interest to mention a little device which had been found useful to ensure accurate mitreing every time the gate was closed, and which he thought went a long way to reduce some of the wear which had been mentioned. Briefly, it consisted of a back stop fastened to one gate ; when the gates were closed, the last few inches of closing of the left-hand gate were effected by the right-hand gate bearing on the stop. The stop acted as an automatic aligning device, bringing the gates into perfect mitre every time.

There was one other point where it would be of value if the Author would amplify a little the description given in the Paper, and that was under the heading " Repairs to Roller Paths." It was not easy to follow the arrangements shown in *Figs 12*. In the Rawlbolt hole under the grouting pipe a wooden plug was shown, and, if Mr Jellett's interpretation of the procedure and the drawing was correct, the whole space above the plug was ultimately filled with grout. How, then, was it possible subsequently to get the holding-down bolt into the holes where the grout had been applied ?

Mr J. S. S. Ramsay recalled a debate [6] which had been held at the Institution in 1948 on the proposition that mitre gates were becoming obsolete. He observed that the present Paper would have made a valuable contribution to that debate.

He noted that after 30 years working of the Entrance Lock to the King George V docks with three pairs of gates it had been decided to provide a fourth pair, to be kept as spares, in view of the possibility of serious damage

[6] A. B. Porter and F. M. Easton, " That Mitre Gates are becoming obsolete." Maritime and Waterways Engg Div. Debate, Instn Civ. Engrs, 1948.

to the gates. He thought that that was a splendid idea, and he hoped that it would be widely adopted.

He thought that it was a tribute to the designers, the builders, and the maintenance staff that so many gates had been kept in working use for so many years without any major repair or overhaul—or at any rate, he had thought so until he had read the section of the Paper headed " Gate Structure " ; he had then realized that, as in the case of so many bridges, it was sheer force of habit that kept those gates in use.

It was stated, on pp. 45 and 46 of the Paper, that at Tilbury it had been possible to dock a pair of gates and to complete their overhaul, without interruption by shipping calls on the dry dock, " in the comparatively short period of 4 months." Did that explain the difference in cost between the lower and upper entrance gates given in Table 3 ? Four months might be a comparatively short period, but he would not consider it a short period for the complete occupation of a large dry dock. Incidentally, it would be interesting to know whether the dock charges were included in the cost of repairs.

On p. 84 of the Paper, the cost of " Handling, floating craft, etc.," was given as 10 per cent of the total cost of repairs. Did that include all labour and supervision in docking, removing, and re-stepping gates, and all charges for hand tackle, tools, and floating craft ? The Port of London Authority were well equipped for such operations and had an experienced staff regularly engaged on them. The figure of 10 per cent would probably be higher for dock authorities less well equipped.

On p. 51 of the Paper it was stated that " When the floating crane was handling large gates . . . in the outer position, the programme was arranged to ensure that the gate was stepped at or near high water and before the tide ebbed to a level where the loss of buoyancy caused the load on the crane to approach the safe working limit." Had it been possible, as a result of any untoward delay, for the tide to fall to a level where the loss of buoyancy would cause a dangerous overload on the crane, and, if so, how would that situation have been dealt with ?

On p. 51, reference was made to difficulties caused by mud when landing gates. It was stated that, in one case, the weight of the gate plus the mud was more than the crane could be expected to lift with safety, and secondly, the gases present in the mud were a source of danger to the workmen. No one who had been in contact with Thames mud would question that statement. Thirdly, when the tidal chamber contained a significant quantity of mud and, for one reason or another, the buoyancy chamber was filled with water instead of air, what the Author described as a " heavy condition " arose, which placed an undue load on the anchors, pivot, rollers, roller path, and on the structure generally. Had the measures taken to keep the gates clear of mud been effective ?

The wear on the greenheart heel-posts, detailed in Table 1, raised some questions : 2 inches of wear at the bottom of the heel-posts would result

in a gap of 4 inches between the bottom of the mitre-posts. How was that gap filled ? Had it been done by adjustment of the rollers, or, where there were no rollers, by fitting tapered pieces on the mitre-posts ? In either case it would be difficult to maintain as good a contact between the mitre-posts as on new gates, and that was bound to result in excessive pressure at points of contact ; the sill prevented the mitres closing at the bottom and distortion occurred, causing unknown stresses in the gate. Leakage to a greater or less degree, depending on the success in closing the gap, was inevitable. With a lock entrance that was not a serious matter, but it made one wonder if all the time and the labour spent on the initial fine dressing of the new gates to a tolerance of 0·004 inch was really necessary. The fine dressing was responsible for a considerable part of the cost of greenheart.

The information given by the Author under the heading "Heel-Posts, Mitres, and Sills" well deserved study. Greenheart was becoming an increasingly difficult problem. That fact had been borne out by a recent Paper [7] on the reconstruction of Greenwell's No. 1 Dry Dock at Sunderland, where a new type of meeting-face had been adopted. The Author of that Paper had stated that, by using the method he described, all expensive granite construction for the entrance had been eliminated, together with all the problems of accurately setting the granite quoins and the precise dressing of the meeting faces. Furthermore, it became possible to abandon the use of greenheart completely, which in turn eliminated one of the frequent sources of leakage at dry-dock entrances. The Port of London Authority had had no choice in the matter so far as existing dock entrances were concerned. The cost of finished greenheart—£12 per cubic foot—was very high. Perhaps the Author would confirm the accuracy of the statement on p. 77 that it cost more to dress a flat surface than to dress a curved surface, because Mr Ramsay's experience had been the reverse of that. An allowance of 3 inches on the finished dimensions for dressing seemed extravagant, and 1 inch was usually ample.

It would be interesting to know the cost of repairs to the masonry, which presumably was not included in the figures given in Table 3.

Although he appreciated the advantages gained by adopting the U-strap type of anchorage, Mr Ramsay pointed out that, if the U-strap was not properly adjusted it had to support the weight of the gate in the open position, and that might produce excessive bending stresses in the legs of the strap. Also, in the open position, the bearing area was much reduced, and that would lead to excessive wear. Satisfactory lubrication was extremely difficult. In the case of the drop-over plate and link anchorage, maintaining the anchorage in correct adjustment was a remarkably simple operation. The drop-over plate could have an anti-friction

[7] Harry Ridehalgh, "The Reconstruction of Greenwell's No. 1 Dry Dock and Ancillary Works at Sunderland." Proc. Instn Civ. Engrs, Part II, vol. 2, p. 321 (June 1953).

bush, good lubrication could be provided, and the bearing area of the gate in the open position was greatly increased as compared with the area provided by the U-strap.

Table 3 emphasized the ever-increasing level of costs, and it was to be noticed that, in some cases, repairs cost almost as much as new gates, if the Author's estimated figures were accepted. Incidentally, it would be interesting to know whether those estimates had been made at a time corresponding to the repairs, because otherwise they would not be comparable. Mr Ramsay thought that the estimates were all on the light side. If allowance were made for the scrap value of the old gates, there was not much difference between the cost of repairs and the cost of new gates; so that perhaps when materials became more plentiful and quicker deliveries could be offered it might pay to scrap the old gates rather than repair them.

Mr G. A. Wilson observed that much of the Paper might serve to show designers where improvements were needed. On the other hand, those 56 pairs of gates had been working without any great interruption for 20, 30, and in one case 40 years, and, with such a record, change in design could not be lightly undertaken. The type of improvement which seemed worthwhile was the sort of thing mentioned in Mr Ridehalgh's Paper, on the reconstruction of Greenwell's Dry Dock, where the cost of dressed granite and greenheart had been avoided.

A number of design points had been mentioned in the present Paper, and it might be useful to list the alterations which had been made in a recent pair of gates for the Gallions lower entrance, which was being reconstructed. At the time of making out the contract, the quoins had not appeared to require renewal, so no alteration in the design of heel-post was considered.

Two obvious changes had been made. The old riveted gates with rollers and roller paths had been replaced by welded buoyant tank gates, and ram operation was being provided instead of chains, because the latter tended to wear the sills and restricted the useful depth of the entrance. Both hydraulic and electric operation of the rams had been considered. Whilst hydraulic machinery could be very satisfactory, occasionally it required fairly heavy repairs; and when it was found that the electrical machinery had a lower first cost, it had been decided to try electrically-operated ram gates. The design was not yet complete, so that he did not propose to commit himself any further on that.

The heel castings had had dowels added to them and were welded as well as bolted so that they would not work loose. The lozenge in the gudgeon of the heel casting was going to be held in with set screws, because sometimes when a gate was lifted the lozenge fell out and had subsequently to be sought for in the mud at the bottom of the lock. The heel-post bolts were welded into the gate to prevent leaks; it was considered that the prevention of leaks was more important than being able to tighten the bolt during the life of the heel-post.

The gangways and stools on top of the gates were bolted to angle cleats welded to the upper deck so that they could be easily removed before lifting the gate, since damage frequently occurred if that were not done. Tidal scuppers had been increased in size considerably and put on the pressure side of the gate, so as to get the maximum buoyancy and to reduce the collection of mud. Provision was made for fixing cover plates over the scuppers when it was desired to float the gates. Trunkways had been placed as near the pressure side of the gate as possible to assist in balancing when the gate had to be removed by flotation.

No pipes had been taken through the interior decks because they could cause leaks and maintenance was costly. Where it was necessary to have pipes they went down the trunkway. In the new gates they were only for blowing air to clear the tanks. Hydraulic ejectors were not fitted since it had been found that portable electric pumps lowered down the trunkways would do the job, and when not wanted in the gates were useful for other purposes. Sluices on the gates had been omitted because they were not particularly successful in cleaning mud from the aprons and required expensive maintenance.

Plate and link anchorages had been adopted in preference to rigid brackets, because, as Mr Ramsay had indicated, the rigid brackets endeavoured to restrain forces which were so great that eventually the brackets loosened the masonry. It was better to have the link and plate, which was positive and adjustable without making other difficulties.

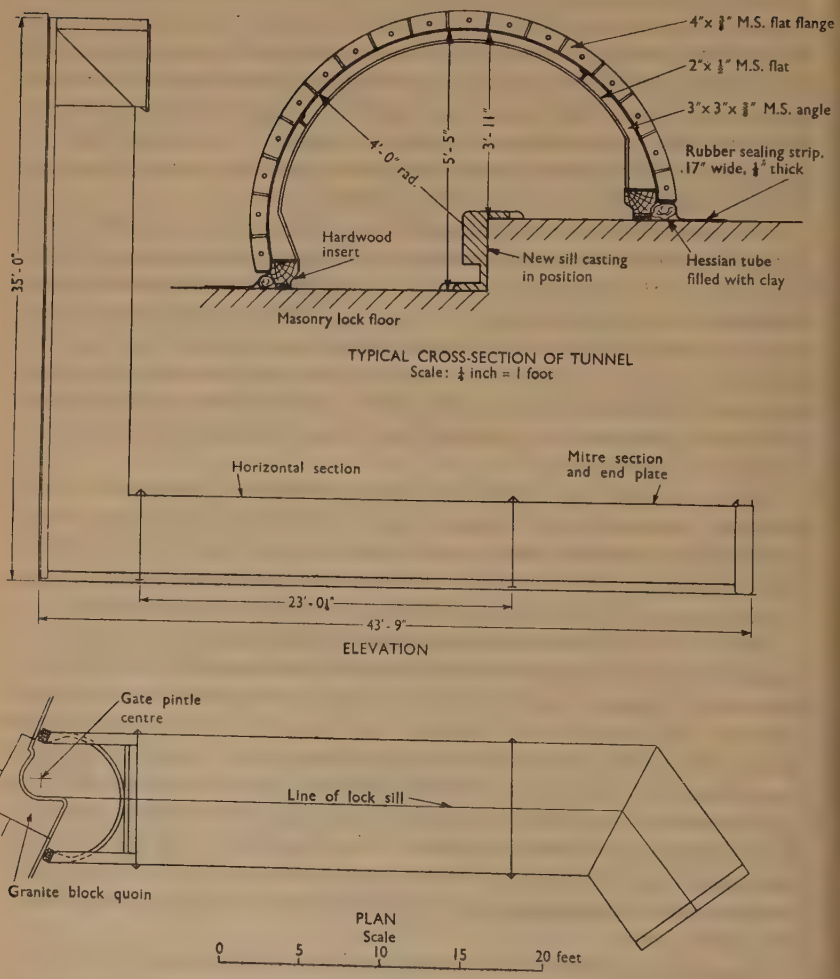
Mr V. H. Tripp described a new development which had recently been successfully carried out in connexion with the renewal of the ship lock gates at the Regents Canal Dock, which had only one entrance to the River Thames. The under-water work of renewing the pintles, dressing the quoins, and putting in a new cast-steel sill in place of the existing timber one had had to be carried out with the minimum of interference to shipping.

The method chosen had involved a limpet dam with a tunnel extension. That idea had been developed and a design prepared, from which a scale model had been produced and tested successfully in a tank. The results had been quite encouraging, and the fabrication of the full-size job had then been put in hand.

Essentially the limpet and tunnel consisted of five sections (*Figs 26*): (1) a vertical section which was the limpet proper, which had flanged connexions for attaching the tunnel at the top and bottom, so that it could be used on either side of the lock; (2) a removable diaphragm, incorporating an 8-inch sluice valve, to enable the limpet to be dewatered independently before the tunnel sections were attached; (3) a straight tunnel section; (4) a mitred tunnel section; and (5) an end cover plate shaped to fit the step of the sill.

The tunnel extended approximately 6 feet beyond the mitre so that a little more than one-half of the sill would be dewatered at a time, and then it was transferred to the other side. The limpet weighed 7 tons and was

Figs 26



LIMPET AND TUNNEL

35 feet deep; the tunnel was of semicircular section, 8 feet in diameter and had a total length of 43 feet. In all, about 14 tons of steel were used. It was constructed of welded mild steel, ¼ inch thick, suitably stiffened, and terminated with 8-inch-by-3-inch channels welded along the edges. Stiffened flanges were provided at the extremities for bolting the sections together.

Oak timbers were fitted into the channels to carry packings to form the

joints with the masonry. The limpet joints were made with tallow-dressed spun yarn fastened to the timbers with canvas strips, and the horizontal tunnel joints with ½-inch rubber insertion bonded to the timber. The vertical joints between the sections were sealed with ½-inch rubber insertion bonded to the steel.

All joints proved satisfactory except the rubber bonded to the timbers, which showed signs of failure after a fortnight's immersion. When the apparatus was transferred, the rubber was found to be quite loose. Soft-wood strips were substituted with satisfactory results.

The final sealing of the tunnel had been carried out by means of 5-inch-diameter "sausages" of puddle clay in hessian bags. Those had been placed along the outside edges of the timber, and 18-inch-wide strips of rubber insertion, fastened to the timbers, had been drawn over them, and laid on the floor of the lock.

Dewatering was effected by means of a 7-inch submersible pump, and subsequent seepage was dealt with by a small hydraulic ejector. At the first attempt to dewater the limpet, it had been found that the upstream leg of the limpet was sliding inwards along the wall of the lock, the maximum movement being at the bottom, diminishing to zero at the top. There had been no movement of the downstream leg.

The limpet had been withdrawn and found to be distorted. It was restored to shape by jacking, and additional stiffening was welded on.

The behaviour of the model had induced undue reliance to be placed on the friction between the limpet and the wall. To remedy the defect, stiffened channels were bolted to the lower half of the lock wall by divers, so that the limpet could be lowered over those channels with ½-inch clearance to the inside of the limpet channels. No difficulty was then experienced and the dewatering was accomplished.

The sill castings, which weighed 2 tons each, were placed along the sill and the tunnel sections positioned over them and bolted into place.

The floor of the lock was reasonably level. Any major depressions, such as the grooves hollowed out by the gate-operating chains and the recesses from which the old roller paths had been removed, were filled in with concrete by divers before the sections were placed in position. Minor variations were filled by means of specially prepared softwood packing-pieces, which were placed in position by divers. The resulting seating of the tunnel was excellent, and after the sealing had been carried out the leakage was very small.

As a safety measure, short channel stops were bolted to the floor of the lock close to the diaphragm before removal of the latter, and also at intervals along the tunnel after dewatering before work in the tunnel was commenced.

A further safety measure consisted of anchoring the tunnel and limpet by means of chains and turnbuckles attached externally to the sections and to ring bolts set in the walls and floor.

An electric derrick crane of 7 tons capacity at 90 feet radius handled all lifting from the lock side.

Inside the tunnel a small travelling cradle with three-dimensional motion was used to handle the sill castings. (*Fig. 27.*)

The working height in the tunnel was 5 feet 6 inches and the work proceeded with the minimum of discomfort. The limpet and tunnel being at atmospheric pressure, with no air-lock, unhampered movement was possible and the compressed-air tools working inside prevented the air from becoming foul.

A heavy fender of steel framing faced with timber and bolted to the lock side protected the limpet. This reduced the lock width from 60 feet to 51 feet. The dock was able to continue in operation subject to a draft restriction at certain states of the tide ; the depth of water over the tunnel varied from 28 feet to 5 feet.

Disregarding the initial setback, the time taken to set, dewater, and seal ready for working was 1 week for each side, including time required for setting the fender.

Fig. 28 showed the completed sill inside the tunnel.

Mr I. S. S. Greeves observed that one of the main factors contributing to the successful use of the limpet dam at the Surrey Docks had undoubtedly been the provision of ample pumping capacity during the initial stages of securing it. The rapid draw-down inside the dam had created sufficient difference in pressure to enable the canvas and oakum strip to take up the contours of the dock wall. It was surprising how tight the seal became when the full hydrostatic head had been developed. All too often when the creation of a seal in a cofferdam depended on the establishment of difference in pressure, those on the site tended to lose heart because they did not employ sufficient pumps in the initial stages.

The Author had perhaps been fortunate in that in most instances he had encountered little wear on his granite hollow quoins. The re-dressing of granite quoins to fine tolerances might be a very costly business in present times, particularly since there was a shortage of skilled craftsmen capable of carrying out that work.

In the case of extensive repairs to quoins, did the Author consider that it might be satisfactory to put them back in a high-grade concrete ?

On p. 75, mention was made of the use of the Port Authority's diving bell. One of the engineer's difficulties in marine work was very often the interpretation of a diver's report. In very silty water, particularly such as was found in the Thames, the diver might be working purely by touch, and by the time the messages got back to the engineer it often became little more than a question of wishful thinking. Where the bottom of a dock had to be examined, the diving bell was quite a simple and effective means by which the engineer could go down and see for himself what had to be done.

With the Port of London Authority engineer, Mr Greeves had spent

Fig. 27

Interior of Tunnel showing Travelling Cradle

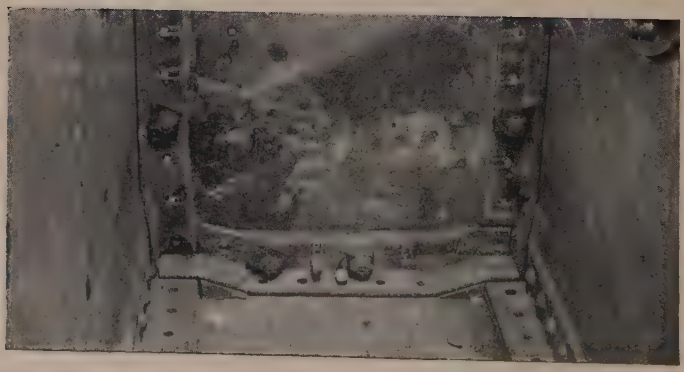
Fig. 28

Completed Sill inside Tunnel

Fig. 29

Fractured Heel-Socket Casting

Fig. 30



Use of Machined Tapered Packings to avoid Joggling Difficulty

2½ hours one afternoon at the bottom of the St Katharine Dock entrance trying to find out the cause of the trouble and how best it could be overcome. Working in a diving bell in that way might be a little hard on the ears as compared with tunnelling in compressed air, because there could be sudden changes of up to ½ lb. per square inch or so, owing to the crane driver picking the bell up and dropping it down again when one was moving about the bottom. By blowing a quantity of compressed air out below the bottom of the bell it had been possible to exclude almost all the water, and the bottom could be made quite dry.

Mr Greeves then showed a short film illustrating some work carried out in the St Katharine Dock entrance with the aid of a diving bell.

Mr J. A. Fisher, referring to the all-welded limpet dam shown in *Figs 19*, pointed out that the rubber seal which was shown as the joint between the lock wall and the dam had not been used or adopted in that form, and had been replaced by canvas and oakum in the more normal way.

On p. 45, the gates installed at the King George V entrance to the Royal Docks were referred to as weighing 350 tons. That was the weight of the original gates, which had been of riveted construction; the all-welded gates which had recently replaced them weighed between 240 and 250 tons. That showed the tremendous advantage to be gained by the use of an all-welded gate—a saving in steel of almost 100 tons in a single gate.

Referring to the method described by the Author for handling gates, Mr Fisher said that he fully appreciated that, in the Port of London, the authorities had been faced by the problem of a large number of old gates which had not been attended to for many years, owing to the war, and which were not very reliable from the point of view of being handled by means of flotation. Moreover, the Authority had been favourably situated in having not only the four wreck lighters but also the 150-ton crane to handle those gates. In the circumstances, the arrangements adopted had been admirable and had enabled a most comprehensive programme to be carried out in a comparatively short time.

Once the gates had been repaired and were in an improved condition, however, he felt that there would be opportunity to consider the method of controlled flotation referred to by the Author on p. 49. That had not so far been used, but Mr Fisher felt that it would be possible at a later date and might effect a considerable saving of money and time in the handling. Controlled flotation was, in his view, a perfectly well tried method. The Dover Train-Ferry gates, weighing about 540 tons, had been well handled and quite successfully floated into position on a slipway in a comparatively short time.

As a further example, Mr Fisher mentioned that a new gate for the Royal Docks had been launched at Thornaby-on-Tees on the afternoon of Friday, 23 May, 1952, towed to London, held in position against the wall of the dock, prepared, and actually stepped on Wednesday, 28 May, 1952—a period of only 5 days (including a week-end) between leaving the slipway

at Thornaby and putting the gate on its pintle ready for immediate use in the mid-position of the King George V entrance. That gate displaced, in its buoyant condition, about 250 tons.

On pp. 53 and 54, the Author had given some interesting figures for the wear-down on the greenheart heel-posts and, on p. 74, the wear on a granite quoin, taken from examination of gates recently repaired. Had the Author had an opportunity to ascertain whether or not it had been possible to arrive at any relationship between the wear-down figures and the actual operation of those gates, that was to say, the number of times which they had been operated since their previous repair ? It might be possible for the Port of London Authority to give such figures, which would be interesting because it was always a difficulty, when considering the replacement of gates, to know whether it would be necessary or not to re-dress the hollow quoin. If it were known that a gate had operated so many thousand times since the quoin had last been inspected, it might be possible to say roughly what wear-down on the granite quoin might be expected.

Mr F. M. Easton said that the comments which he had to offer arose from experience in the maintenance and renewal of dock gates in the South Wales ports during the period from about 1923—shortly after the South Wales ports had been brought under Great Western Railway ownership—until they passed under the control of the Docks and Inland Waterways Executive.

His first comment related to the statement on the first page of the Paper that " it is considered advisable to overhaul lock gates at intervals of not longer than 15 years." Why take out a gate simply because it had been in service for 15 years ? The factors which determined whether a gate needed to come out were the efficiency of its design and the service conditions which it had to sustain, with the result of those conditions in corrosion and wear. The deciding factor was almost always the condition of the greenheart. A gate was seldom or never taken out in order to paint it. It might be necessary to take it out for roller trouble or heel-socket failure, but usually gates had to come out when the sealing timbers showed, by leaking, that something had to be done. The period might vary from 10 years to 30, or even longer.

With regard to the Tilbury gates shown in *Fig. 3* (*a*), he considered that the subdivision was overdone. All that a gate needed by way of watertight subdivision was a watertight deck to separate the buoyancy chamber from the tidal chamber and a central watertight diaphragm to make it possible to keep ballast water in the heel end or the mitre end when handling the gate for removal or stepping ; further watertight subdivision was not worth while. He agreed with Mr Fisher that controlled flotation was quite a good method of handling gates ; it was cheap and quick, but it did need experience, especially under tidal conditions.

On p. 50, the Author had stated that " The gates were also cleared

beforehand of as much mud as possible and wooden plugs were inserted in the scupper holes of the tidal chambers . . . ." No mud should be allowed to accumulate in a gate. It was not possible to stop the mud accumulating in the tidal chamber of an inner gate, where the dock water was at constant level, if the tidal chambers had the scuppers on the back—the pressure side. If, however, those scuppers were moved to the face of the gate the tidal chamber would become accessible and it would be possible to keep the gate clear of mud by periodically flushing out with fresh water, and at the same time to keep an eye on the condition of steelwork or ironwork. Friction between timber heel-posts and masonry hollow quoins would provide ample margin against hydrostatic uplift.

On p. 52, reference was made to the gases present. That had been mentioned by Mr Ramsay, but Mr Easton thought that Mr Ramsay had misinterpreted the point which was made there. The gases which the Author had in mind were, Mr Easton believed, those which accumulated in a sealed chamber and which were dangerous to men before the gas could be cleared out. That had been the experience at Barry Docks in 1925, when a man had been killed by the explosion of a mixture of methane (marsh gas) and air present in a buoyancy compartment which had not been opened for many years. That gas was produced by the decay of marine organic matter which got into the chamber in minute quantities, possibly through seam laps and bolt holes under pressure of the tides. As a result of that accident, the practice had been instituted in the South Wales docks of keeping all gates open for access and inspection, and providing ventilation by the installation of pipes where necessary. No compartments were left permanently sealed.

The wear referred to on p. 53, was common in Mr Easton's experience and just about normal for the conditions mentioned, but in the 90-foot entrance at Cardiff the maximum wear on the heel-posts in 43 years had not exceeded $\frac{15}{16}$ inch, and there had been similar wear in the 100-foot entrance at Newport. That low rate of wear could be attributed to the unusually large radius of the heel-posts—2 feet in each case. Referring to the clearance between the bottom gudgeon (the heel socket) and the pivot, he agreed that a 1-inch clearance was normal and proper for the larger entrances, 60 feet or more in width. For an entrance of 50 feet or less at least $\frac{3}{4}$ inch clearance was necessary, so that as the timber wore down, the socket should not press and wear on the pivot. The base of the heel socket should be kept a like distance, radially, inside the heel-post timber so that the socket should not bear into and score the hollow quoin. The projecting rivet heads which had been mentioned should not project; they should have countersunk heads, only slightly raised, to the proportions specified by Lloyd's. Ordinary structural countersinking was far too shallow for watertight work.

The trouble referred to on p. 55 had also been experienced at Swansea Docks, with badly supporting deck angles kneed to clear the corner angles

at the heel end, and considerable racking and loosening of rivets had been found. The supersession of riveted by welded design would obviate the considerable difficulties which might otherwise be expected through the increasing scarcity of competent angle-smiths who could produce those single and double joggles, and similar work.

Mr Easton presumed that the small amount of corrosion and the good behaviour generally of the gates in the Port of London docks were partly attributable to the low salinity of the water. He imagined that the water was nearly fresh, at any rate by comparison with the ordinary sea-water at the South Wales docks, except where there was a fresh-water feeder into the dock. It appeared also that the Port of London engineers had not been troubled in the way that the South Wales docks engineers had been by *teredo* infestation.

He thought that the idea for taking the weight at the top of the spear rod shown in *Figs 11* (*c*) was one of the most interesting and novel features in the Paper. It gave a better chance of adjustment than the customary gibs and cotters, which usually seized up so that one could not move them when one wanted to do so. But why have any rollers at all, especially on inner gates ? If gates were designed with medium preponderance of weight over buoyancy, the need for rollers disappeared. Those highly expensive fittings were needed only where there must, of necessity, be an unusually heavy preponderance to keep the gates steady while being operated in a very heavy swell, as at Plymouth Docks.

He wished to express his approval of the drop-over type of anchorage, which he thought was the best possible means for keeping the non-roller gates in proper adjustment. The anchorage should be so designed as to eliminate the possibility of thrust coming upon it from the top gudgeon, and so tightly screwed up as to keep the heel-post bearing well into the hollow quoin over as great a part of its length as possible. By that means, wear on the greenheart would be minimized.

Mr Easton then displayed a series of lantern slides illustrating the points he had raised. Two of his slides, reproduced as *Figs 29* and *30* respectively, showed a heel-socket casting which had fractured under heavy service conditions ; and a way of obviating the joggling difficulty, using machined tapered packings which enabled a " lazy joggle " to be employed.

Mr M. W. Leonard described, with the aid of lantern slides, some remedial work which had been undertaken in 1950 to the sill of the inner lock gates to the main entrance to Avonmouth Dock. The main sill was about 8 feet wide and 4 feet deep. A head of 32 feet of water was acting from the back. The bedding joint had become very defective, and those responsible had been very concerned about the erosion effect of the water coming through in very large quantities.

The sill was about 75 feet across. It was exposed in the spring tides, and there had been a limited time in which it had been possible to get to

work, and a certain number of physical hazards in getting on the sill and working. The men had to work in diver's suits without helmets, immersed right up to their shoulders in some cases. The idea had been to plug or caulk the open joint with hardwood wedges. The gap had been about $\frac{1}{2}$ inch in places for the wedge to enter, and the depth of the void anything from a few inches to 5 or 6 feet. It had been decided to work from the heel across to the mitre, and progressively stop the water interfering with the men as they worked.

A jack-hammer was used to bore $1\frac{1}{4}$-inch holes into the bedding joint, in which grout pipes were inserted at 6- to 8-foot intervals. An attempt had been made to use fantail grout pipes to get a wider insertion at the joint, but he did not think that that had any beneficial effect at all. Each of the grout pipes had been fitted with a cock, so that it was possible to isolate the work as they proceeded. After the first pipe had been connected up, the lock was filled and a diver worked his way along linking up the hose with the remaining pipes. As each was grouted up under pressure he turned the cock off to stop any back flow.

After grouting along the face, the sill was exposed and it was found that the remedial work had been effective. Some holes had, however, been drilled round the back to grout up any void not filled from the front. They then had a stiff face to force the grout against. About 9 tons of grout had been pumped in, together with 4 tons of very fine sand dredged from the Bristol Channel. It had taken about 37 shifts to complete the work.

The Author, in reply, said that many fundamental points about the design and operation of lock gates had been raised by the discussion. With regard to the suggested 15-year period for the overhaul of lock gates which had been referred to by Mr Jellett the Author hoped that that did suggest some change of heart, because it was important that the maintenance of lock gates should be carried out on a properly planned basis at not too infrequent intervals. Experience seemed to suggest that if the gates were taken out for inspection, repainting, etc. after a reasonable number of years the heavy cost of deferred repairs would be avoided.

The repair of gates in the vertical position had taken longer because of the interruptions caused by periodic flooding required by the normal dry-docking programme. It was true that there was some reduction of overhead work when the gates were in a vertical position but there was also the disadvantage of having to erect heavy and complicated staging around the gate. The Author agreed with Mr Jellett that the whole period of one tide was not necessarily required for the operations, but it was necessary to complete the work within that time, not only for the safety of the gate and plant employed but also because generally the operation of the lock concerned could not be interrupted for longer without delay to shipping. In practice, only the outer gates were affected by tidal conditions. During the repairs to the roller path described, three adaptors and grout cocks were fitted to each half of the path. The pump was connected to one cock, and

discharge pipes connected from the other two cocks to the lock sides acted as observation points. The plugs could be withdrawn and bolts inserted before the grout set finally. Air pressure was applied later, as a test, by withdrawing two of the intermediate bolts.

Mr Ramsey had referred to the debate[8] on mitre gates which had been held in 1948. The Author felt that there was still a future for mitre gates, particularly in river entrances where limitations of space had to be taken into consideration. The gates at the Tilbury entrances were of very different design and structure and an appreciable amount of work had been put into the machinery, which probably accounted for the difference in costs. For various reasons the dry dock at Tilbury had been available for use and the Author did not think that any dock dues had been charged against the gates, but the use of all plant, etc. had been included in the item for handling. If, as the result of some accident, the tide had fallen to a dangerous level when handling outer gates, each gate could have been lowered to the bottom of the lock and, provided the floating crane was securely moored during the low-water period, everything would, he thought, have been all right.

Where the wear on the heel-posts had been appreciable tapered pieces had been fitted to the mitre-posts and, where possible, the rollers had been adjusted. With reference to the cost of greenheart, the cost of dressing the heel-posts on a pair of gates repaired by the Author had been £3 15*s.* per cubic foot, compared with £4 0*s.* per cubic foot and £3 2*s.* per cubic foot for the mitre-posts and sills respectively. An idea of the cost of repairing the masonry could be obtained from the costs quoted for operating the limpet dam, which included all the plant and labour on the job.

The Author had had roller gates in mind when discussing the U-strap type of anchorage, but he knew of gates which had worked for nearly 50 years with that type of strap without mishap, although the gate rollers had been worn until there were large flats on them. There was much to be said in favour of the more rigid type of anchorage, for cases of damage to heel-posts had occurred when the flexible type of plate anchorage had allowed the timber to come into contact with the projecting nib on the hollow quoin.

The Author considered that the repair costs given in Table 3 could be taken as comparable, but he did not think that old gates had much scrap value, for removing and cutting up an old gate was in itself a very costly business. Whether new gates should be ordered instead of repairing the old ones had depended on a number of factors. When many of those gates had been handled the delivery period for new gates was 2 years or more and there had been a world-wide shortage of steel at the time.

There had been some improvement as a result of the measures taken to reduce the accumulation of mud in the tidal chambers. Mr Wilson had referred to new welded gates ordered for the lower Gallions entrance and

[8] See footnote 6 on p. 89.

Mr Fisher had mentioned the saving in steel in the all-welded gates as compared with the riveted gates at the King George V entrance, and it was remarkable that that could amount to as much as 100 tons in a single gate of that size.

The Author had been concerned with the repairs to existing gates, but Mr Wilson had given details of a number of new features which had been incorporated in the design of the new gates now under construction for the Port Authority. It was interesting to note that amongst them was the adoption of electrically operated rams, because the Author had had experience of that type of machinery about 20 years previously at a dry dock in the West Country, and so far as he was aware there had never been any trouble with it. Operating conditions at entrance locks were, however, very different from dry docks, and it would be interesting to observe how the electrically operated machines and gearing behaved under such conditions. To substitute electric pumps for hydraulic ejectors in existing gates it would probably be necessary to make some alteration in the design of the trunkways to facilitate the handling of the pumps.

The repairs to the sill at the Regents Canal Dock entrance, which Mr Tripp had described, were of great interest because the Authority were faced with similar problems at some of the older entrances in the Port. Mr Tripp could be congratulated on a very fine achievement, but he had been fortunate in that the depth of water over the sill gave a substantial margin for the vessels regularly using the lock. In some of the entrances to the Authority's docks there was, on occasions, very little depth of water to spare and therefore the solution adopted by Mr Tripp was not possible. The position was made worse in some instances by the fact that no alternative entrance was available.

The Author was interested to note that it had been found that the friction between the limpet and the wall had been insufficient to prevent it from sliding, because that seemed to justify the struts which had been employed in the limpet used at Surrey Docks and described in the Paper.

Mr Greeves had stressed the importance of ample pumping capacity to provide the rapid draw-down required to set up the initial seal between the limpet and the wall. The Author would agree that the success of the limpet was largely attributable to the pumping capacity available. The use of concrete for repairing hollow quoins had been limited to small patches. If a high-grade concrete were to be used extensively for that purpose, careful attention would have to be paid to the materials used. It was doubtful, however, whether anything approaching the finish possible with granite could be obtained on a large scale, so that more rapid wear of the heel-post timbers might result.

Mr Greeves had mentioned the sudden variations in pressure caused inside the diving bell by the operation of the crane. There had also been an uncomfortable increase in the pressure when the bell had been used in places where a considerable depth of silt had accumulated. If the bell

were lowered too suddenly under such conditions there could be a momentary intensification of the pressure to a dangerous degree. There had been some difficulty in manipulating the diving bell over sills where the invert of the lock had appreciable curvature.

The Author was grateful to Mr Fisher for pointing out that the original design of the all-welded limpet had been altered to include a canvas and oakum seal instead of rubber. He agreed that when the gates had been restored to an improved condition it might sometimes be more convenient to handle the gates by controlled flotation. But, as Mr Ramsey had remarked, the Port of London Authority were well equipped with floating plant, and if it was available the Author felt that it was better to use it and thus eliminate a certain amount of risk during transit of the gate from the repair site to the lock through a dock crowded with lighters, whose movements were sometimes quite unpredictable. It was worthy of note in that connexion that the new gates fitted at the new Waterloo river-entrance[9] at Liverpool were not fully self-docking because suitable floating plant was available for gate-docking operations. Mr Fisher had suggested that it would be of value if the wear-down of the heel-posts and hollow quoins could be related to the number of times the gates had been operated since their previous repair. That information could be made available from the Dockmaster's logs at each lock, for now that each gate had its own identification number, a corresponding log of its movements could be kept. The operation of the gates varied considerably. Internal gates such as those between a basin and the main dock might be operated only once or twice on each tide, and always with the water at the impounded level. A check had been made on the use of the Greenland entrance, where, on an average, about twelve locks were made on each tide. During 15 years therefore the gates were operated nearly 200,000 times.

That fact, the Author thought, partly answered Mr Easton's question as to why it was suggested that gates should be taken out of service after about 15 years. He had derived much benefit from Mr Easton's own valuable Paper[10] on the subject and welcomed his remarks. He (the Author) thought that the degree of sub-division in the Tilbury gates might be an advantage when handling the gates by controlled flotation, because it limited the quantity of " free water " in the gate which would make for better stability.

Mr Easton appeared to be in favour of putting the scuppers on the non-pressure side of the gate, but it was noticeable that that was a controversial point among engineers. No doubt local conditions and the position of the entrance did have some bearing on the point.

The Author had always thought that the precautions taken to test for gas were in some cases a little unnecessary and it was comforting to have

[9] R. A. Stephenson, " Design and Construction of the Waterloo River-Entrance." Maritime Paper No. 15, Instn Civ. Engrs, 1950.

[10] F. M. Easton, " Dock Gates." *Structural Engineer*, vol. 16, p. 359 (Nov. 1938).

confirmation that accidents might have been avoided by taking that amount of trouble.

It was true that the salinity of the Thames water was of a low order, typical values being 1·6 to 6·0 parts per 1,000 of chlorine at 7 miles below London Bridge and 0·1 to 2·3 parts per 1,000 in the Upper Pool, depending upon the season and state of the tide, but he believed that the quantities of impurities present were giving the responsible authorities some cause for concern. He confirmed that they had no trouble, at present, from marine borers in the Port of London.

In recommending the drop-over type of anchorage Mr Easton had referred to the importance of keeping the anchorage tightly screwed up. In practice, that might give more trouble from a maintenance point of view than the more rigid bracket type, which the Author had referred to earlier in his reply ; but, as Mr Wilson had stated, it had now been decided to adopt the former type at the reconstructed Gallions lower entrance.

The work which Mr Lennard had described was typical of the problems with which dock engineers were faced at some of the older works for whose maintenance they were responsible, and it was instructive to learn that the technique employed on that occasion had been effective.

RAILWAY ENGINEERING DIVISION MEETING

24 November, 1953

Mr W. K. Wallace, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 51

"Bridge Investigation, Ceylon Government Railways, 1949–1951"

by

Conrad Gribble, O.B.E., M.I.C.E.

SYNOPSIS

The investigation into the strength of the bridges on the principal railway lines in Ceylon was undertaken by the Author on behalf of the Crown Agents for the Colonies, at the request of the Ceylon Government. The object of the enquiry was to ascertain to what extent the bridges could safely carry existing locomotives, and what strengthenings or renewals would be required in order to render them safe for existing loads and also for the heavier engines which may be required in the future.

Stress and deflexion recorders were used in order to measure the effects of the heaviest engines in use, when travelling at speed, as shown by the vibrations of the girders produced by the "hammer-blows" of the engines and by lurching.

On certain parts of the railway system the effects of centrifugal forces on bridges situated on sharp curves were investigated, as were the horizontal oscillations of girders carrying straight track, attributed to the absence of sway bracing. The effects of "hammer-blow," lurching, and horizontal oscillations are shown in diagram form in relation to the spans of the bridges tested, and comparison is made between the total effects observed in the investigation and those represented by certain impact formulae in use, including the recommendations in the Bridge Stress Committee's Report of 1928.

Introduction

The investigation into the strength of railway bridges in Ceylon, which is the subject of this Paper, was undertaken by the Author on behalf of the Crown Agents for the Colonies in response to a request made to them by the Ceylon Government in May 1948.

In this Paper the Author describes the methods adopted in conducting the enquiry in order to deal with the various problems which presented themselves, and he also gives an account of some of the strengthening works which have been carried out on the bridges in the island from time to time.

Description of Railway and Bridges in Ceylon

The first railway in Ceylon was opened in 1865. After a somewhat keen controversy a gauge of 5 feet 6 inches was adopted, that gauge having been chosen for the Indian railways.

Fig. 1 is a map of the Ceylon railway system with the date of the opening of each section. In addition to the broad-gauge railways, two lines having a gauge of 2 feet 6 inches were built, but one of these and part of the other have since been taken up.

Of the present total route mileage 810 miles are of broad gauge and 85 miles of narrow gauge. The railways are mainly single line with passing places at most of the stations. However, the 45 miles from Colombo as far as Polgahawela on the main line to Kandy and the 17 miles from Colombo to Panadure on the Coast Line have been doubled.

There are in all about 1,150 underbridges (excluding all less than 6 feet in span), of which 1,030 are on the broad-gauge lines and 120 on the narrow gauge. The few overbridges which exist were not included in the investigation.

The great majority of the bridges were built with open floors, the track being carried on longitudinal or transverse timbers attached directly to the girders. Bridges built before 1900 were of wrought iron, structural steel having been first introduced into Ceylon about the beginning of this century.

Standard drawings were made for spans of various lengths, there being comparatively few types and a considerable number of bridges of each type.

Plate girders were adopted for bridges of spans not exceeding 60 feet but, for the longer spans, double-intersection Warren trusses were employed, with few exceptions.

A number of these truss bridges have since been renewed with steel N trusses or with plate girders, but many of the original wrought-iron bridges remain, and these formed the subject of the recent investigation.

The gradients and curves on all lines, except on part of the main line to Kandy and the " up-country " line to Bandarawela and Badulla, are comparatively easy, but on the latter branches, which pass through very hilly country, many bridges are placed on sharp curves, the maximum span of these being 80 feet. These bridges are " deck " spans for reasons which are obvious, but the longer spans carrying straight track are " through " or " semi-through " spans, the depth of the girders being about one-tenth of the span. In general, only spans of 120 feet or longer have overhead bracing and none of the wrought-iron bridges has any horizontal diagonal sway-bracing.

In recent years a large number of the smaller spans have been replaced by bridges carrying a ballasted track, and there are quite a number of

Fig. 1

MAP OF CEYLON RAILWAY SYSTEM SHOWING DATES OF OPENING FOR TRAFFIC

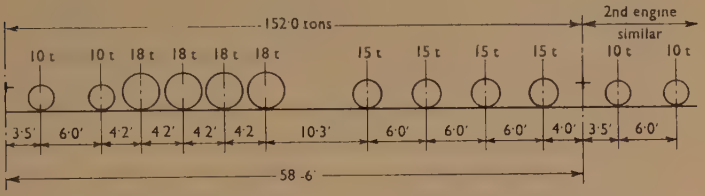
rail-decks. The oldest spans still in existence were erected about the year 1880.

Standards of Loading adopted in Design of Bridges

No records exist of the standard of loading adopted in the design of the older bridges nor of the limits of stress assumed in their design. About 1910, a standard loading diagram for the calculations for all future bridges was adopted by the then consulting engineers in Westminster, and this was worked to until the inception of the recent investigation.

This loading, referred to as " M.P.L." (maximum probable loading) is shown in *Fig. 2*. It was based on a type of locomotive which was used

Fig. 2



Maximum Probable Loading, 1910—21 B.S. Units (Approx.)

in India, and approximates to 21 B.S. units of loading. Its effect is greatly in excess of that of any engine used, or even considered for use, in Ceylon, and it does not represent a type of locomotive which would be generally suitable. Its fixed wheel-base would be too long to negotiate the sharpest curves on the " up-country " line, and its wheels would be of rather small diameter for travelling at speeds up to 50 miles per hour, which are required on the other lines. The railway authorities in Ceylon, during the progress of this investigation, decided that 16½ B.S. units of loading should cover any types of engine which may in future be introduced into the island, and this standard is now assumed in all calculations made to ascertain the capacity of the bridges to carry future loads.

It is not known, however, at the present time, whether future engines will be steam-driven or diesel-electric. An order has recently been placed in the United Kingdom for twenty-five diesel-electric machines, whose weight does not materially exceed that of the heaviest existing steam engines. The axle loads of these engines are shown in *Fig. 3*. Three of them are already in use.

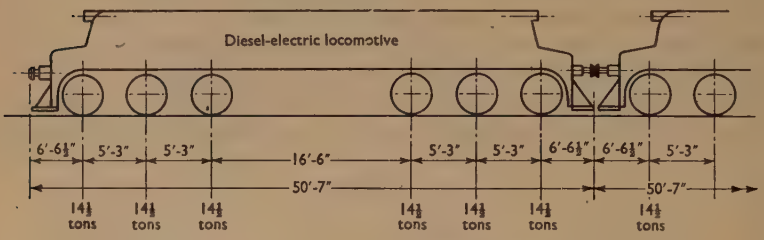
The engine now used in Ceylon which causes the maximum total effect on bridges is that known as the B.1 class, and its axle loads and hammer-blows are shown in *Figs 4*. One of these engines was used in all the tests carried out in the investigation.

The new diesel-electric engines which produce no hammer-blow will

have an advantage when compared with the existing steam engines, because they will produce small impact effects.

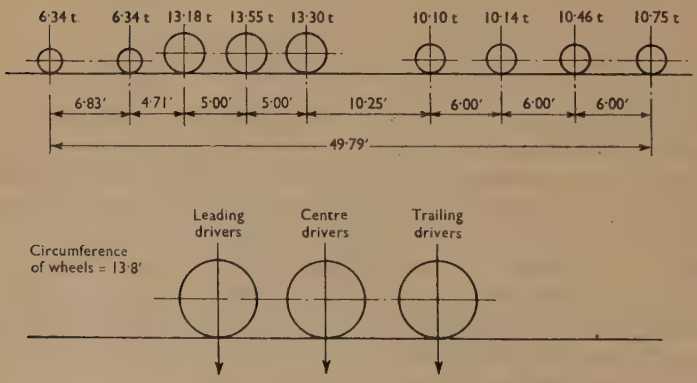
In tabulating the results of the calculations of the stresses caused by various standards of loading, it has been necessary to show the stresses

Fig. 3



Wheel Spacing and Axle Loads for Diesel-Electric Engine

Figs 4



	Leading	Centre	Trailing	Whole engine
Hammer blow at 1 r.p.s : tons . .	0·163	0·006	0·157	0·326
,, ,, ,, 20 m.p.h. tons .	0·736	0·027	0·709	1·472
,, ,, ,, 25 ,, ,, .	1·151	0·042	1·108	2·301
,, ,, ,, 30 ,, ,, .	1·660	0·061	1·599	3·320
,, ,, ,, 35 ,, ,, .	2·26	0·08	2·17	4·51
,, ,, ,, 40 ,, ,, .	2·94	0·11	2·84	5·89
,, ,, ,, 45 ,, ,, .	3·73	0·14	3·59	7·46
,, ,, ,, 50 ,, ,, .	4·60	0·17	4·43	9·20

Axle Loads for Class B.1 Engine

caused by future engines of the 16½ B.S. units of both two-cylinder steam and diesel-electric types, and to indicate what strengthenings or renewals would be necessary before either of the two types is put into service.

Previous Enquiries

On two occasions, in 1910 and 1918, the consulting engineers, Messrs Gregory Eyles and Waring, made reports on the strength of the then-existing bridges, giving advice in some cases as to the need for strengthening works. They classified the bridges in respect of their capacity to carry maximum existing loads (M.E.L.) and maximum probable loads (M.P.L.) respectively. Their calculations were based on information sent to them by the Chief Engineer and not on any personal inspection by an engineer from England.

These reports did not state the actual stresses calculated to exist in the bridges reported upon, nor did the consulting engineers reveal what limits of stress they considered to be safe. In many instances they stated that bridges were, or would be, " fully stressed " or " overstressed " by M.E.L. or M.P.L. but it has not been possible to ascertain from the reports what stresses, expressed as tons per square inch, they had in mind when using these terms. Nor is it possible to determine what, if any, allowance for impact effects was made in their calculations nor how they discriminated, in dealing with various bridges, between those on lines where speed is limited to 20 miles per hour and those where speeds as high as 50 miles per hour may occur.

More recently, in 1936, the Crown Agents were asked to report on the strength of the bridge over the Kelani river near Colombo, a structure of importance, to which reference is made later in the Paper.

The recent enquiry not only marks the first occasion in which a deputation from the United Kingdom has visited Ceylon to investigate the problems at the site, but it is also the first time in which modern stress- and deflexion-measuring apparatus has been used there to assist in determining the effects of moving loads.

Strengthening Schemes to provide for Heavy Engines

The programmes of strengthening and renewal of bridges which have been carried out from time to time have been fairly extensive.

All the bridges between Colombo and Nawalapitiya on the Main Line, with the exception of the Kelani river bridge, have been brought up to the standard of M.P.L. (1910).

On the same line, between Nawalapitiya and Bandarawela, a scheme of strengthening was put in hand about the year 1920 in connexion with the introduction of " Garrett " engines. This occupied several years, but the scheme did not increase the strength of the bridges as a whole to the standard of M.P.L. (1910) which would have materially increased the cost.

At the end of the 1939–45 war, however, a decision was made to proceed with a scheme of renewals of bridges below the standard of M.P.L. (1910) with the object of gradually bringing up the principal lines to that standard of strength. That scheme, if completed, would have involved a very large expenditure indeed.

Orders were placed in the United Kingdom for the construction, in the first place, of about twenty bridges ranging in span from 20 feet to 140 feet. These spans were designed by the Crown Agents who also obtained tenders and supervised their manufacture. The largest span, 140 feet, had been erected before the recent investigation was completed and the Chief Engineer is now proceeding with the erection of the remainder. The adoption of the lower standard of loading—16½ B.S. units—for future designs and the recommendations made by the Author as the result of the investigation have of course very materially affected the future policy of the railway administration in regard to the bridge renewal programme.

Instruments Employed in the Investigation

In order to carry out the desired tests, suitable stress- and deflexion-recording apparatus was needed. It was hoped that instruments similar to those used by the Bridge Stress Committee in the United Kingdom during the years 1923 to 1927 could be employed. The apparatus actually used at that time could not be spared for the work in Ceylon and enquiries regarding the possibility of purchasing suitable instruments revealed that the " Fereday-Palmer " stress recorder is no longer manufactured.

Owing to the limited resources of Ceylon for repair and maintenance of delicate scientific instruments, the apparatus to be employed had to be of a simple and robust type, and the following eventually became available :

One Fereday-Palmer stress-recorder kindly lent by Messrs Rendel, Palmer and Tritton.

One Cambridge stress-recorder lent by the Nigerian railway.

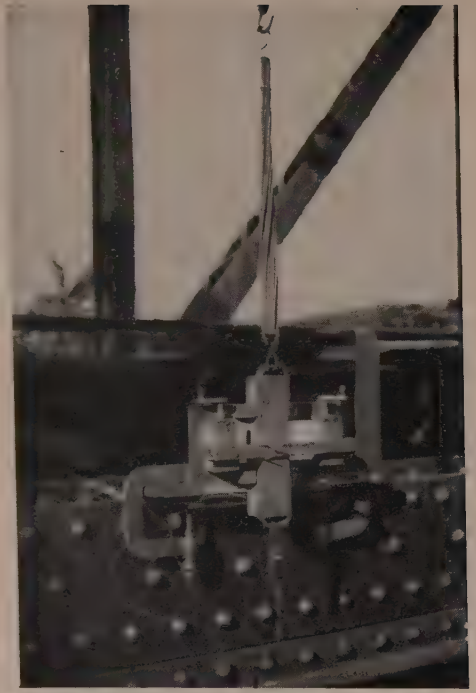
Two Cambridge " vibrographs."

Two " La Touche " strain meters (lent by British Railways), which do not yield a graphic record but must be read visually after each test, and are of somewhat limited usefulness.

The Fereday-Palmer stress recorder and the La Touche strain-meters were the only instruments available to the Author when he left for Ceylon in August 1949, and the remaining apparatus was received some months later.

On arrival in Colombo it was decided to design and make a recording deflectometer of simple type in order to tide over the period pending the arrival of the vibrographs from England. This instrument, shown in *Fig. 5*, was designed by Lt.-Col. Eagan (to whom reference is made later),

Fig. 5



THE "EAGAN" DEFLECTOMETER

Fig. 6

FOUR INSTRUMENTS ON BRIDGE

Fig. 7

Locating Position of Maximum Hammer-Blow

Fig. 8

Rail Contact Instrument for Locating Position of Maximum Hammer-Blow

and came to be known as the " Eagan " deflectometer. It recorded deflexions at full size by means of a pencil which scribed on a moving-hand-wound spool of paper. It has been usually employed in recording both vertical and horizontal deflexions but it lacks any " time " or " location " indications. The Cambridge stress-recorder and the two vibrographs were designed to record electrically " time " signals from a common clock registering tenths of seconds and " location " signals from rail contacts, the wiring for these indications and for starting and stopping the films in the instruments having been prepared in Colombo.

It should be mentioned that each of the three Cambridge instruments records by means of a stylus on an uncoated celluloid film.

The Fereday-Palmer instrument, which records photographically, has its own time clock registering half seconds and, since it was connected to the rail contacts, it was possible to synchronize the records obtained from the four recording instruments. This stress recorder was hand-operated, and not controlled by the master switch which governed the movement of the films in the other instruments.

In *Fig. 6* are shown four instruments on a plate girder arranged to record vertical and horizontal deflexions and stresses in both top and bottom flanges.

The datum for measuring deflexions was provided by means of a taut wire fastened to a heavy weight dropped to the ground, generally the bed of a stream, immediately below the instrument, the wire being tightened by means of a stranded rubber spring fastened to a point on the girder above the instrument.

For certain tests the exact location of the positions of the maximum force of hammer-blow on the span was of importance and these points were determined by means of an imprint in a trough of clay made by the end of a small wire fastened to one of the spokes of a driving wheel of the testing engine, and projecting slightly beyond the tread. The position of the wire in relation to the point in the circumference of the wheel where the maximum out-of-balance occurs enables the positions of maximum " hammer-blow " on the bridge to be ascertained and to be subsequently marked on the records of deflexion and stress by means of the indications on the records given by the rail contacts. See *Figs 7* and *8*.

For such an investigation as that now being described, the apparatus employed is generally satisfactory, but the relation between the records taken from the various instruments in point of " time " cannot be determined with a very high degree of accuracy. The indications made by the rail contacts are not precisely simultaneous in all the instruments on account of slight variations in the speed of operation of the various solenoids. To obtain exact synchronization would require a far more delicate adjustment of these coils than is made in the instruments as designed and manufactured. This fact came to light when it was desired to ascertain whether or not the two girders of a through bridge oscillated in

phase and how the instant of maximum amplitude in each corresponded with the instant of maximum " hammer-blow " on each side of the engine respectively. The indications made by the rail contacts in each instrument were found to be insufficiently accurate in respect of timing for a precise answer to this question to be obtained, but this shortcoming did not affect the correctness of the values recorded, nor had it any serious effect on the success of the tests.

Commencement of Enquiry, September 1949

The Author arrived in Colombo in September 1949 with the two assistants who had been specially engaged for the investigation—Lt.-Col. R. H. Eagan, B.Sc., and Mr Bernard Bramall, B.Sc., A.MM.I.C.E. He had also the services of several Ceylonese assistants, one of whom, Mr L. S. De Silva, B.Sc.(Eng.), was available for practically the whole period. The work occupied rather more than 2 years, and ended with the issue of the Final Report in November 1951.

The Author formed the opinion, after considering the objects of the enquiry, and the data available, that the main problem would be to ascertain the magnitude of the impact effects of the engines in use, particularly those caused by hammer-blow.

This resolved itself into a measurement of the " dynamic magnifier " (so termed in the Bridge Stress Committee's report of 1928) for the various bridges to be dealt with.

As has been mentioned, the B.1 class of engine was the most severe in respect of its static weight and hammer-blow and it was decided to use one of these engines for all the tests to be carried out. It was necessary to have the out-of-balance of the testing engine checked and this was done at the locomotive works at Ratmalana. The engine was fitted with a speedometer in order that the driver would be able to regulate the speed in each test to that asked for.

With the bridges taken in hand in the first place the intention was to ascertain the critical speed for typical structures of various spans, where such a speed was attainable, and also the extent of the hammer-blow effect at that speed or at the maximum speed reached, as shown by the amplitude of the vertical oscillations of the girders. These were measured by the deflectometer and also by the stress records in the girders which were simultaneously obtained.

It was not possible to make an exhaustive research into the stresses occurring in all parts of the structures. The instruments obtainable were too few in number for such a programme to be completed in the time available.

In the Author's opinion, a fair estimation of the capacity of the bridges to carry the loads, existing or proposed, could be obtained by the following procedure :—

(1) Inspection of the bridges to ascertain the correctness of the drawings, or otherwise, and to measure loss of section by corrosion.

(2) Calculation, by conventional methods of analysis, of the stresses in the girders caused by the B.1 engine and by the new standard M.P.L., considering each to be a purely static loading.

(3) Measurement by means of suitable apparatus of deflexions and stresses in the main booms or flanges of the girders tested, caused by the B.1 engine moving at slow speed, and comparing the stresses shown on the records with those obtained by calculation.

(4) Further measurement of stresses and deflexions during tests at speeds ranging from low speed to the maximum attainable at the site, in order to determine the critical speed, where possible, and the extent to which deflexions and stresses are increased by the action of the engine's hammer-blow.

(5) Using the information so obtained to calculate the maximum stresses likely to occur in the bridges under the following headings (in addition to dead load):—

(*a*) Live load of engine.

(*b*) Hammer-blow impact at critical or maximum speed.

(*c*) Allowance to be made for lurching and effects of rail joints, etc., in accordance with the recommendations of the Bridge Stress Committee (pp. 124 and 125 of their report). An endeavour was made to check these values by information obtained from the test records.

(*d*) In calculating stresses caused by future diesel-electric engines, no allowance was made for hammer-blow impact.

A certain number of stress readings on web members of lattice girders were taken to determine whether the increase of stresses caused by hammer-blow was in accordance with the theory laid down in the Bridge Stress Committee's report (pp. 66–71). After considering the data obtained, which for reasons already mentioned were not complete, it was decided to make the same allowances in calculating impact effects in web members expressed as a percentage of the live load, as in the main booms.

In the calculation of impact in very short spans and in floor members, hammer-blow effects were considered to be static forces with no dynamic magnification, their value being taken to be the actual forces of the out-of-balance weights at the maximum speed attainable.

A systematic method of calculation was adopted for determining values of the dynamic magnifier based on experimentally ascertained values of deflexions and stresses. This is set out in full in Appendix I.

The spans of the bridges which were tested and the number of spans tested are given in Table 1.

TABLE 1

Span : feet	Number of spans tested	Span : feet	Number of spans tested
150	1	80	8
126	1	60	9
120	3	40	3
100	4	30	1
		10	1

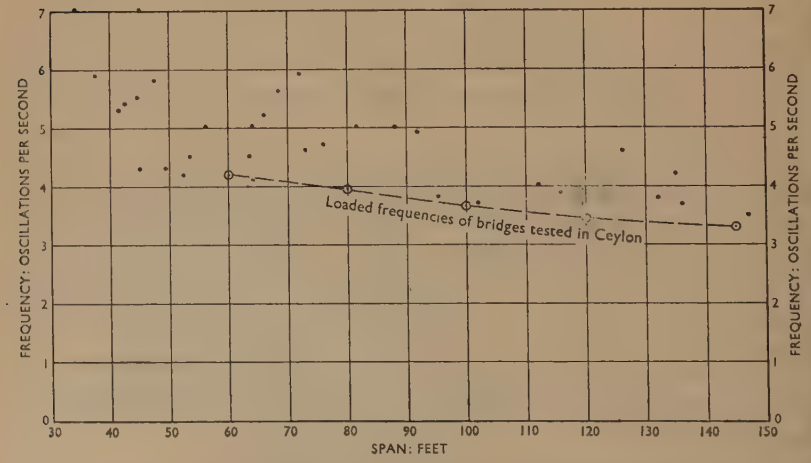
Total number of bridges tested—31

SPEEDS ATTAINED IN WORKING AND DURING TESTS

The speed attained on railways in Ceylon is limited by order to 45 miles per hour although in actual running a speed of 50 miles per hour is reached on certain lines. On the up-country line, speed is limited to 20 miles per hour, but this regulation is sometimes exceeded in actual running by as much as 5 miles per hour on the down-hill journey.

It was thought that the limited speed attained in the island might render it unlikely that, except in the longest spans, the critical speed to

Fig. 9



LOADED FREQUENCIES OF BRIDGES

(The dots represent the loaded frequency of oscillation of bridges tested by the Bridge Stress Committee, 1923–26. The loaded frequencies of bridges tested in Ceylon (1949–51) are shown in comparison).

create the greatest oscillations would be reached, this opinion being based on the data given in the Bridge Stress Committee's report concerning the frequency of oscillation of the bridges tested in the United Kingdom.

The natural frequencies of these bridges when loaded by an engine are shown in *Fig. 9* (spans up to 150 feet) and, in comparison, are shown the loaded frequencies of bridges tested in Ceylon. The latter bridges have, on the whole, lower frequencies than those in the United Kingdom, and this fact is partly accounted for by their being of wrought iron, whereas nearly all the bridges tested in England were of steel.

The diameter of the driving wheels of the engines used in Ceylon is about 4 feet 6 inches and this is smaller than that of express engines in the United Kingdom. With engines of the types ordinarily employed on fast trains in Britain it would not have been possible to attain critical speeds in many bridges in Ceylon since the limits of permissible speed would have been considerably exceeded.

The tests showed that, for the majority of the bridges of 80 feet or more, the critical speed was reached by the testing engine and was within, or not greatly in excess of, the speed usually attained.

Kelani River Bridge, Colombo

The Author was asked to consider the existing restrictions on the Kelani river bridge as a matter of importance and urgency.

The bridge, which is illustrated in *Fig. 10*, facing p. 128, carries the Main Line over the Kelani river about 3 miles from Colombo, and consists of six lattice-girder spans of 80 feet and twelve plate-girder deck spans of 25 feet (six at each end). It was built in 1881 for a double line, although the railway was not duplicated until 30 years later.

Before the second line was brought into use the consulting engineers were asked for advice concerning the capacity of the 80-foot spans to carry safely the double track and their report was adverse to the proposal, and during the whole life of the bridge single-line working has been in operation in the section occupied by it.

In 1936, the Crown Agents were asked to reconsider the matter and their reports were based on information furnished by the Chief Engineer in Colombo in the form of drawings of the bridge and data concerning its condition.

Complete calculations were made in London relating to the stresses in all parts of the structure produced by the various types of engine in use, and as a result extensive strengthening works were found to be necessary and were carried out.

The Crown Agents, however, were not prepared to recommend that, even after these works had been completed, the bridge should be opened for double-line traffic, but they agreed that the speed restriction then in force of 4 miles per hour for all trains should be raised to 25 miles per hour,

that being the highest speed considered to be permissible having regard to the lay-out of the tracks at each end of the bridge.

Specimens of wrought iron from the Kelani bridge were at that time sent to London for metallurgical examination. The tests carried out showed that the metal was of excellent quality and had not suffered any deterioration from fatigue. The Author suggested, however, that the metallurgical tests should be repeated, making use of specimens of wrought iron taken from the original rail bearers which had been removed during the strengthening works referred to and which were still available. The results were similar and just as satisfactory as those obtained in the earlier tests.

It is evident from perusal of documents in Ceylon that the engineers responsible in the past for the maintenance of the bridges in that country, and of Kelani bridge in particular, had fears that the wrought iron of which they are composed might have suffered deterioration by repeated loading during the passage of time, and that the maximum stress which they should be allowed to bear should be rather severely limited.

The metallurgical tests, however, carried out in 1936, and repeated in 1950, were completely reassuring, and the consulting engineers who conducted them informed the railway engineers that they had no reason whatever to fear any deterioration in the quality of the metal.

The Author has not been able to trace any record of a failure of any member of a bridge in Ceylon due to fatigue.

When one considers the frequency of rail traffic in the island and the moderate stresses which have been found to exist in practically all the bridges under present-day loads, there does not appear to be strong reason for expecting fatigue failures.

The specimens sent to London in 1936 withstood 10 million repetitions of $\pm 7{\cdot}5$ tons per square inch stress without any signs of cracks.

During the whole time that the Kelani bridge has been in use there have probably not been more than perhaps one million crossings of the bridge by locomotives. The calculated stresses under the worst conditions do not exceed 6 tons per square inch, the range of stress being less than 5 tons per square inch, which may be compared with a range of 15 tons per square inch used for the tests in London.

After considering these circumstances and having in mind also certain rules recommended for use on British Railways, the Author advised the engineers in Ceylon that no restriction in the running of engines need be enforced where the maximum tensile stress in the wrought iron did not exceed 8 tons per square inch, after making full allowance for impact effects. He suggested, however, that opportunity should be taken to carry out metallurgical tests on specimens of wrought iron taken from the bridges which are to be renewed during the present programme already referred to, in order that future evidence can be obtained regarding the soundness of the material in the bridges still remaining in service.

The tests carried out on the Kelani bridge, which marked the commencement of the experimental work in Ceylon, included deflexion and stress measurements with the instruments immediately available, the object being to ascertain the effects of the B.1 engine at various speeds up to the maximum attainable at the site.

Although the prescribed maximum speed for ordinary traffic is, as has been mentioned, 25 miles per hour, a speed of 40 miles per hour was actually reached during the tests. The calculated natural frequency of the 80-foot span when one track was loaded by the engine was 3·78 cycles per second, corresponding to a speed of 36 miles per hour. The tests showed that the oscillations of the bridges caused by hammer-blow increased with the speed and appeared to be at their maximum at about 36 miles per hour.

The calculated frequency with both tracks worked simultaneously was, however, 3·38 cycles per second, corresponding to a speed of 32 miles per hour. At this speed, the hammer-blow would have a force of 80 per cent of that at 36 miles per hour. It is not likely that such a speed would be attained simultaneously by a train on each track, and, as a matter of fact, it was not practicable to carry out tests with an engine on each track with any hope of success on account of the difficulty of arranging that the engines would cross the bridge with their hammer-blow in phase. Experience obtained during the tests in England had fully demonstrated this difficulty. Although a speed restriction of 25 miles per hour was in force it was decided to calculate the stresses in the bridge for a speed of 30 miles per hour and to assume that both tracks were simultaneously and similarly loaded. It was also assumed that the " dynamic magnifier," as ascertained during the tests with a single engine at that speed, should apply to the hammer-blows of both engines which should be assumed to be crossing the bridge with their hammer-blow in phase. For the rare and improbable occasions on which two trains might produce maximum combined bending moment at the same instant as maximum hammer-blow in phase and at critical speed, a moderate factor of safety would probably suffice to protect the structure against any possible damage throughout overstress.

The calculated stresses in the Kelani bridge based on the above assumption did not reach the proposed limit of 8 tons per square inch in tension, and the Author was able to report that there were no reasons for continuing the restriction of double-line working which might therefore be instituted forthwith if the management so desired.

In making the final calculation of stress, allowance was made for lurching and rail-joint effects in accordance with the Bridge Stress Committee's recommendations, and it should be appreciated, in regard to these factors, that their inclusion in the total estimated stress pre-supposes that their effects coincide in point of timing with those of hammer-blow.

The Author was asked also to say under what conditions, if at all,

engines of 16½ B.S. units could be allowed to work over the Kelani bridge. The results of the calculations were as follows :—

(*a*) With one track loaded, either steam or diesel-electric engine of 16½ B.S. units could safely be employed.

(*b*) With both tracks loaded, diesel-electrics could be permitted but not steam-engines, since the stresses they would cause would be in excess of the permissible limit.

Matale Line

When the bridges on the main line to Kandy were strengthened several years ago to bring the line up to M.P.L. standard, the portion beyond Kandy to the terminus at Matale was not dealt with, and the heavier engines, such as the B.1 class, have not been allowed to work there. On this branch, speed is limited to 20 miles per hour. The Author was asked to ascertain what renewals or strengthenings would be required before the restriction of loads could be removed, and also what further work would be necessary to allow engines of 16½ B.S. units to be put into service.

The longer spans on the Matale branch are generally similar to those on other branches over which the B.1 engines work, and the restrictions were enforced mainly because of the existence of a number of small spans which were of quite insufficient strength. These have now been renewed.

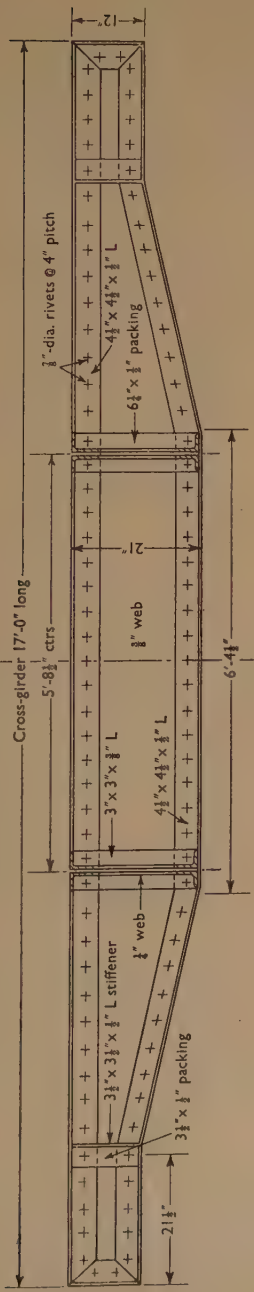
The type of the longer spans generally existing in the island is shown in *Fig. 11*, facing p. 129, and attention is drawn to the design of the cross-girders which are " fish-bellied " in elevation, and are very shallow at their ends.

Calculations of the stresses in these cross-girders showed that they were sufficiently strong, so far as their flanges were concerned, to take all existing loads, but the ordinary methods of calculation adopted indicated that the stresses in the rivets connecting the web to the flanges greatly exceed safe limits.

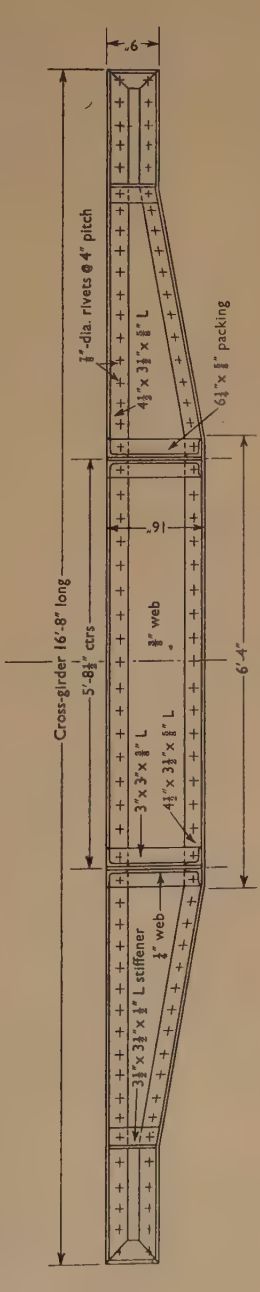
It was realized at the same time that, although on the Matale line axle loads exceeding 9 tons were excluded, precisely similar bridges on other lines were regularly loaded by axle loads of 13½ tons and no signs of distress have been observed. It was considered prudent, before permitting heavier engines to work over the branch, to have specimen cross-girders tested to destruction in order to ascertain whether in point of fact the riveting at their ends is the weakest part of the girder and what load the girders should support before these rivets began to fail.

Two girders shown in *Figs 12* were dispatched to Middlesbrough where they were tested to failure in Messrs Dorman Long & Co.'s test house in April 1950. The girders were laid on their sides and subjected to horizontal thrusts at two points corresponding to the positions of normal vertical loading. The stronger girder failed by the fracture of a flange angle through rivet holes on the tension side. The smaller girder failed by buckling of the web at the points of loading.

Figs 12



CROSS-GIRDER NO. 1



CROSS-GIRDER NO. 2

Scale: ⅛ inch = 1 foot

CROSS-GIRDERS TESTED AT MIDDLESBROUGH, APRIL 1950

Neither girder showed any signs of overstress in the riveting although the stresses which theory would indicate as probable under the loads ultimately applied might be expected to produce a failure of the material in bearing and shear.

The tests were of value in demonstrating that these wrought-iron cross-girders have ample reserves of strength to carry the loads now in use. Table 2 gives particulars of the girders and the loads applied.

TABLE 2

	Test No. 1	Test No. 2
	ft. in.	ft. in.
Length of girder	17 0	17 0
Span between centres of bearings	15 1	15 1
Distance between loading points	5 6	5 6
Depth of web : centre	1 9	1 4
ends	1 0	9
	tons	tons
Total load at failure	90	50
Total load finally	103½	55½

Fig. 13, facing p. 129, shows the second cross-girder after the test. The principal structure on the Matale line is the bridge over the Mahaveli river at Katugastota. This includes four 124-foot lattice-girder semi-through spans which are generally similar to many other bridges of similar span in the island, but differ from the usual type in not possessing overhead bracing, and in the location of the cross-girders at the mid-points of the panels. Since the cross-girders are supported at the centres of the lower-boom members, local bending moments are induced. Where this occurs in other bridges, steps have been taken to remedy the defect by providing hangers supporting the cross-girders by connecting their ends to the intersection points of the web members immediately above them.

It was considered that if such hangers were added to the bridge in question and if suitable horizontal diagonal sway bracing were provided below the floor system, as well as struts between the top booms, the bridge would be sufficiently strong to support not only all existing loads, but future loads up to 16½ B.S. units. The Chief Engineer has accordingly carried out the strengthening works recommended.

COAST LINE

The Coast Line from Colombo southwards to Galle and on to Matara, a total distance of nearly 100 miles, was constructed between the years 1874 and 1895. Many of the bridges have since been renewed, but a number remain, and are now about 60 years old. The line has been doubled for

a distance of 17 miles from Colombo, and all the bridges on the doubled portion are of steel and of full strength. The bridges on the coast lines have suffered considerably more from corrosion than those on the inland branches, where the atmospheric conditions are less injurious, but they have been periodically repaired by replacing heavily corroded members either by riveting or by welding. Steel bridges erected about 40 years ago, to replace earlier wrought-iron structures, have suffered more severely than older wrought-iron bridges still in existence, and in one steel bridge the majority of the rail bearers have been completely renewed. The bridges are regularly and frequently tarred, and have been maintained in good condition though the work has involved much expense.

The maximum speed permitted by regulations is 40 miles per hour but the Author was advised that a speed of 50 miles per hour might be attained on certain portions. There are numerous curves on the line, over which lower speed limits are prescribed, and in calculating stresses in bridges on these curved portions a speed of 5 miles per hour in excess of that laid down has been assumed.

The longer spans were originally of the type described in the foregoing section on the Matale line. The 100-foot spans are semi-through, that is, without overhead bracing, and the 120-foot spans are through spans with " basket handle " bracings between the top booms. None of these bridges was provided with horizontal diagonal sway bracing. The majority of the steel bridges erected as renewals have been designed for the heavy M.P.L. (1910) loading and are of a modern type.

Typical wrought-iron bridges on this line were tested by the B.1 engine at speeds up to 60 miles per hour and the critical speeds for spans of 80 feet, 100 feet, and 120 feet were ascertained. These tests showed that maximum oscillations were induced at speeds between 30 and 40 miles per hour, but in these bridges the stresses calculated to exist at those speeds were not excessive.

It was found, however, that, at the higher speeds, although the vertical vibrations became less, there were horizontal oscillations of some magnitude which were attributable to the absence of sway bracing.

These bridges would not be strong enough to carry engines of 16½ B.S. units having hammer-blow, and in the event of such heavier loads being introduced, strengthening works would be needed, or alternatively, in certain cases, a rather severe speed restriction would have to be imposed. Since the engines in use have comparatively small driving wheels, any speed restrictions, to be effective, are necessarily severe.

The programme of renewals now in progress, to which reference has already been made, includes the replacement of two steel spans on the Coast Line and two similar spans in the north of the island which were erected some 40 years ago. These, which are all in close proximity to the sea, have suffered very severely from corrosion, and have been extensively repaired from time to time, particularly in the floor systems.

Since these girders were designed for the extremely heavy loading of 1910, and apparently for a moderate working stress, they have, in consequence, a large margin of safety and still retain ample section to render them strong enough for not only all present-day loads, but for 16½ B.S. units.

It has therefore been recommended that these spans should remain, and that the new steel girders obtained for their renewal should be used to replace the older wrought-iron bridges of the same span, which, although not unduly stressed at present, would not be strong enough to carry a heavier loading in future.

Bridges on the Up-Country Line

The bridges on the line between Nawalapitiya and Bandarawela presented a problem which was different from that met on the Coast Line and other branches. On this line, which is about 80 miles long, there are, in addition to many shorter spans, a large number of 60-foot and 80-foot spans carrying the line over ravines, many being on very sharp curves. The curve over one bridge tested was found to be less than 5 chains radius. On the whole of this portion of line a speed restriction of 20 miles per hour is prescribed. The line rises by gradients of about 1 in 45 to a height of more than 6,000 feet above sea level and in doing so winds round the hillsides in a series of horseshoe curves. Owing to the severe speed limitation, hammer-blow impact is not a serious factor. Even if, as has been assumed, this speed is exceeded on occasions by 5 miles per hour, the critical speed for resonance is not reached in any of the bridges which, with one exception, do not exceed 80 feet in span. At 25 miles per hour, moreover, the force of hammer-blow is small. All the spans situated on sharp curves are deck bridges, lattice girders having been chosen for spans of 60 feet and longer, and plate girders for the shorter ones.

There are semi-through bridges of 80 feet span on straight portions of track.

None of these bridges was provided with any horizontal lateral bracing and, as might be expected, the centrifugal forces exerted by a locomotive when travelling at even the limited speed permissible are sufficient to cause considerable horizontal bending moments and lateral deflexions.

The lack of such bracing was pointed out by the consulting engineers in 1918, but for reasons which have not been ascertained, no action in this respect had been taken prior to the recent investigation. About 30 years ago, before the introduction of " Garrett " engines, eighteen 60-foot deck spans were strengthened by the addition of centre girders. These girders had been recovered from bridges on the Coast Line which had been renewed with steel girders, and they were of similar type to those which they were used to reinforce. They not only relieved the main girders of a portion of their load, but they also reduced the stresses in the cross-girders by halving their span.

Notwithstanding the advice of the consulting engineers, in their report of 1918, that it would be impossible by such an arrangement to obtain an even distribution of load between the three girders on curved bridges, the engineers proceeded with their scheme of adding centre girders. In the Author's opinion, the results were very satisfactory, and no doubt the 80-foot spans would have been similarly strengthened if suitable girders had been available.

At the same time, a number of 30-foot and 40-foot plate girders were strengthened by the addition of steel flange plates. The method employed was to construct a new span to replace an existing bridge, and remove the latter to the bridge-yard at Colombo, where the new flange plates were added by riveting. The strengthened span was then used to replace another bridge which was dealt with in the same way, and so on.

It so happened, however, that there were difficulties in removing three of the spans, one of 40 feet and two of 30 feet, and these were strengthened in situ. The work was carried out in stages, one flange plate at a time being removed, and used as a templet to mark off a new steel plate for drilling. The old plate was replaced immediately and bolted up. When the new steel plate was drilled the old wrought-iron plate was again removed and both it and the new steel plate were placed on the girder and again bolted up. These operations were carried out between trains and continued at each of the three bridges until the new plates had been attached to the whole length of the girders on both top and bottom flanges. Finally, they were riveted.

This method of procedure resulted in the flange plates taking little, if any, share in the dead-load stresses which were borne by the web and main angles. " Chromador " high-tensile steel was used for the new flange plates, though it is difficult to appreciate any advantage in employing this material in positions where its superior tensile strength could not be utilized.

Although for long spans such a method of strengthening would be open to criticism, the results in the 30-foot and 40-foot bridges seem to have been quite satisfactory.

The programme of strengthening now in progress, to which reference has already been made, includes a number of bridges on the " up-country " line.

This scheme provided for the complete renewal of one 140-foot span and nine 80-foot spans. The intention was to employ the wrought-iron girders removed from these nine bridges to reinforce an equal number of 80 foot spans, not scheduled for renewal, by converting them into " four-girder " spans. This proposal would, however, leave a number of 80-foot spans to be dealt with by complete renewal or in some other way.

The Author has recommended that the recovered girders should be used to convert the remaining 80-foot spans, all of which are situated on curves, into " three-girder " bridges, and that the additional girders should

be placed, not on the centre line of the bridges, but adjoining the girders on the outside of the curve in each case. Such a method of strengthening would relieve the girders which would otherwise be overstressed by engines of $16\frac{1}{2}$ B.S. units and render the bridges capable of safely carrying such loads.

Tests on the Up-Country Bridges

The tests on the up-country bridges were in the main intended to ascertain the stresses in the girders caused by centrifugal force. Observations showed that appreciable horizontal deflexions occurred when an engine passed at speed. Although the prescribed speed is 20 miles per hour, it was understood that at times a speed of 25 miles per hour is reached, and during the tests 30 miles per hour was exceeded on occasions.

To measure horizontal deflexions required the establishment of a fixed point, close to the bridge, from which readings could be taken, and it was only in certain bridges, where the height of the girders above the ground was not too great, that it was found possible to provide the necessary datum point. This was done by erecting a steel post adjoining the bridge and securing it by guys. The Eagan deflectometer was used to measure the horizontal movements of the girders during passage of engines. The "Vibrographs" were used at the same time to measure the vertical deflexions of the two main girders.

Stress records were also taken on the outer girder of the bridge by means of stress recorders on both flanges.

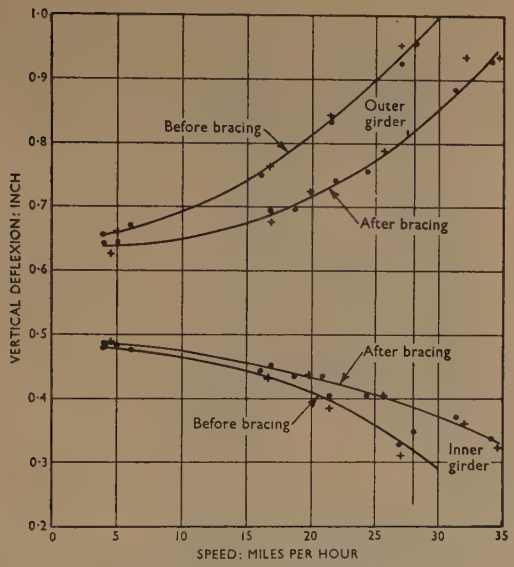
Table 3 shows the results of the tests on a bridge of 80-foot span, situated on a curve of 4·85 chains radius. The speed attained by the engine was 27·5 miles per hour. It was recommended that horizontal diagonal bracing should be added to all the 80-foot spans situated on curves, the bridge tested being dealt with first and then re-tested. Table 3 shows the test results after strengthening. The speed was the same as before.

Table 3

	Before	After
Horizontal deflexion at mid-span : inch .	1·60	0·3
Vertical deflexion of outer girder : inch .	0·95	0·84
Tensile stress recorded in flanges : tons/sq. in.	4·9	3·5
Compressive stress recorded in flanges : tons/sq. in.	5·75	3·4

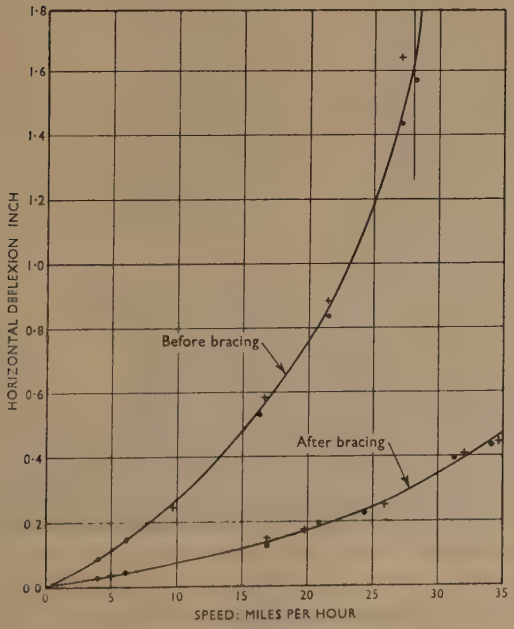
Fig. 14 shows the vertical deflexions of the outer and inner girders at various speeds both before and after the bracing was added ; *Fig. 15* shows

Fig. 14



VERTICAL DEFLEXIONS OF 80-FOOT-SPAN BRIDGE ON CURVE OF 4·8 CHAINS RADIUS

Fig. 15

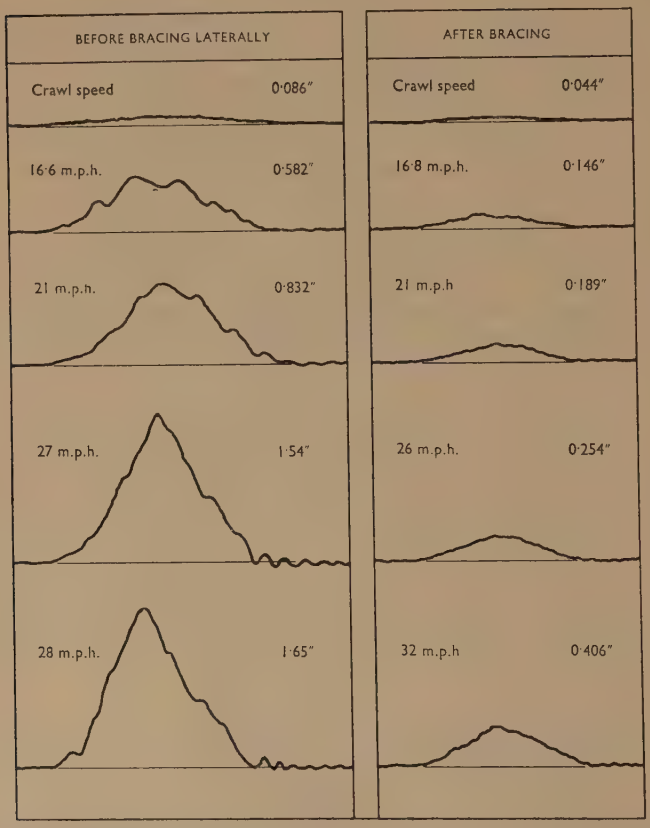


HORIZONTAL DEFLEXIONS OF 80-FOOT-SPAN BRIDGE

the horizontal deflexion of the girders measured at the same time; and *Figs 16* show the actual deflexion records taken before and after bracing.

Tests were also made on one of the three-girder 60-foot bridges which was situated on a 7½-chain curve. As was expected, the horizontal deflexions were considerably less than in the two-girder 80-foot span. A reading of 0·42 inch was recorded at a speed of 27·5 miles per hour.

Figs 16



Horizontal Deflexions of 80-foot-Span Bridge loaded by Test Engine

It was not thought to be urgently necessary at the present time to add lateral horizontal bracing to these three-girder spans since they are much stiffer than the two-girder bridges, but it is recommended that it should be done before engines of 16½ B.S. units are introduced.

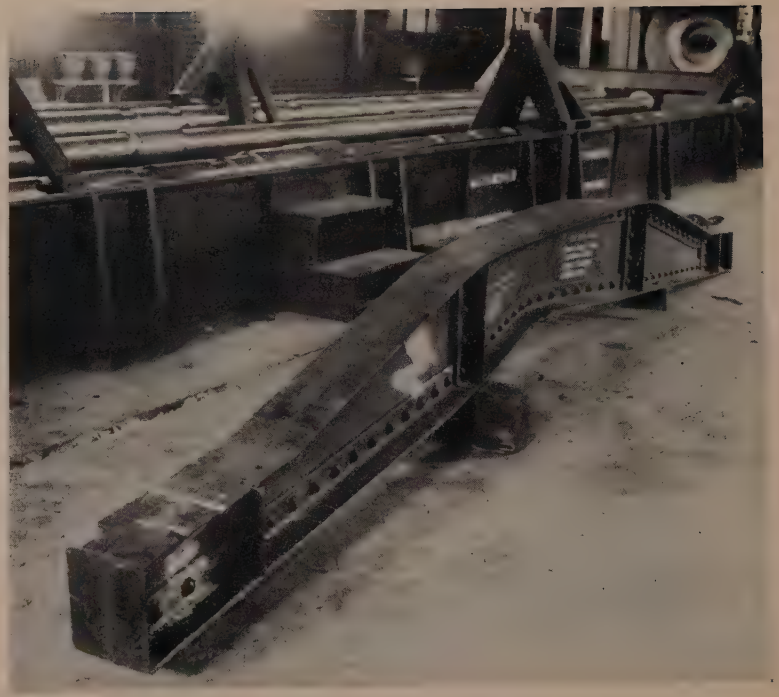
Fig. 10

KELANI RIVER BRIDGE

Fig. 11

TYPICAL THROUGH GIRDER BRIDGE

Fig. 13



CROSS-GIRDER AFTER TEST

Resistance of Bridges to Horizontal Bending

The absence of horizontal diagonal cross-bracing in the bridges carrying sharply curved track does not prevent the structure as a whole from resisting lateral forces to some extent, because the cross-girders, which are riveted to the tops of the main girders, provide some stiffness so long as the rivets remain tight.

The tests on these bridges were intended to ascertain to what extent the structures acted as rigid horizontal frames and in that manner increased the lateral rigidity beyond the horizontal stiffness of the individual girders.

A method of calculation was devised by Mr Bramall, which is set out in Appendix II, where the results obtained by tests on several bridges are shown in graphic form. These indicate that from 50 to 70 per cent of the centrifugal force on the bridge may be considered to be resisted by the structure acting as a rigid frame. In making calculations for the bridges of this type it was assumed that the proportion was 50 per cent. In these calculations consideration was given to the distribution of load between the two girders, having regard to the position of the track relative to each and the superelevation of the outer rail ; this entailed a careful survey of each bridge.

It will be appreciated that the provision of horizontal diagonal sway bracing to these bridges renders the resistance to bending provided by the stiffness of the individual girders a negligible factor.

The investigation showed that the strengthening works undertaken between 20 and 30 years ago had gone a long way towards rendering the line as a whole capable of carrying not only now-existing loads, but also engines of $16\frac{1}{2}$ B.S. units as proposed in the future.

When the present programme of renewals has been completed and the 80-foot spans not entirely renewed have been converted to three-girder bridges as recommended, the line will be strong enough for the heavier engines.

Northern Line

The bridges on the Northern Line from its commencement at Polgahawela to Kurunegala were brought up to the standard of M.P.L. 1910 several years ago and practically all the bridges on the branch to Talaimannar Pier are also of that standard of strength.

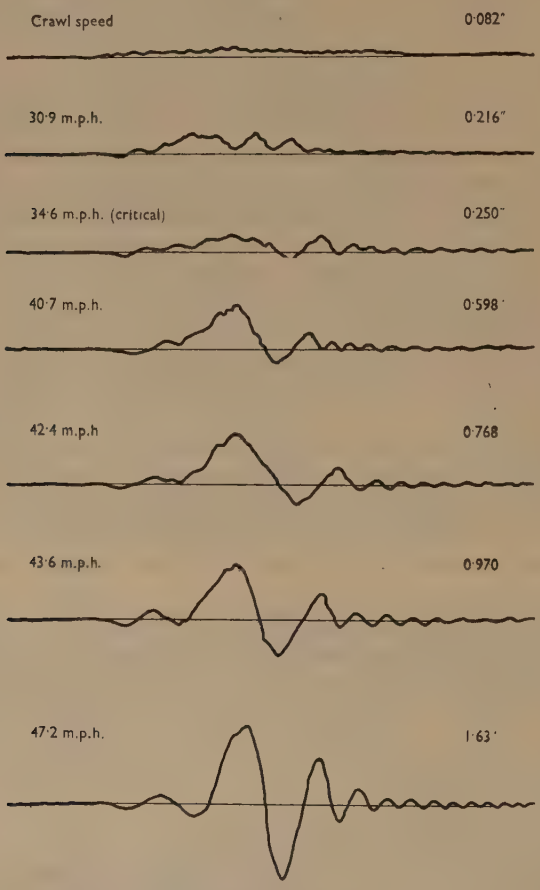
The portion from Kurunegala to Jaffna was under construction when the use of wrought iron in bridges was superseded by the introduction of steel. The bridges built before 1900 are of wrought iron and those erected later are of steel, but were made to the same drawings as before.

The bridges are of the same general type as those on the Coast Line and the conditions are very similar. It was desired to carry out tests at

rather higher speeds, if possible, in order to obtain further data regarding the behaviour of the bridges when so loaded.

It was not, however, practicable to exceed a speed of 52 miles per hour in any of the tests. So far as hammer-blow effects were concerned, this

Figs 17

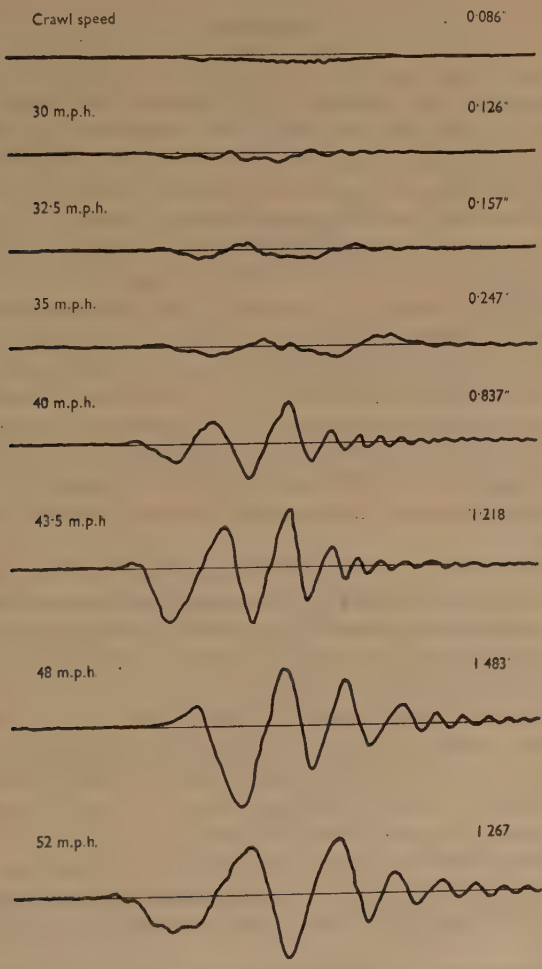


HORIZONTAL DEFLEXIONS OF 100-FOOT LATTICE SPAN

was satisfactory, since the critical speed for the bridges tested was reached and passed. In no case was there any evidence of the existence of a second higher critical speed resulting from the coming into action of the locomotive springs with a corresponding raising of the natural frequency of oscillation of the loaded bridge, although it is reasonable to suppose that at a sufficiently high speed, if such were to be attained, this phenomenon would occur, much as it did in the tests in the United Kingdom.

The tests on the Northern Line, however, did show in a marked degree the existence of horizontal vibrations at speeds higher than those which caused the greatest vertical movements, and these are no doubt augmented

Figs 18



HORIZONTAL DEFLEXIONS OF 120-FOOT THROUGH SPAN

by the lack of horizontal sway bracing. *Figs 17* and *18* show the results obtained in tests on two spans of 100 feet and 120 feet respectively.

At the critical speed for hammer-blow effect the horizontal vibrations are small. They increase rapidly as the speed is raised up to the maximum obtained.

It has been recommended that all the lattice-girder spans should be reinforced with adequate sway bracing as soon as possible. In the meantime a speed restriction to, say, 30 miles per hour would be effective in limiting the horizontal movements.

Lurching

The Bridge Stress Committee recommended that in designing bridges it should be assumed that lurching of the engine on its springs might cause a transference of load from one side of the engine to the other of 25 per cent at the highest speed taken into account in computing live-load effect, that is, 6 revolutions per second of the driving wheels, and that at lower speeds the allowance should be proportionately reduced. It would appear reasonable to assume that the magnitude of possible lurching is more dependent upon the speed of travel of the locomotive than on the revolutions of the driving wheels. In the case of the express engines employed in the United Kingdom the maximum effect would be related to a speed of 80 miles per hour or thereabouts. For the engines used in Ceylon, however, the corresponding speed would be only about 50 miles per hour.

The factor of lurching is of considerable importance when dealing with deck bridges where there are two main girders spaced, as a rule, not more than 10 feet apart, and in the case also of rail-bearers. It does not so seriously affect the stresses in the main girders of through bridges nor in cross-girders.

It is not very easy to measure the effects of lurching from records which show also the oscillations of the girders caused by hammer-blow, and they could more easily be ascertained by tests with engines which have no out-of-balance, for example, four-cylinder engines such as those used in the United Kingdom, if such were available.

It has, however, been possible to deduce from the records taken in Ceylon certain values of lurching by eliminating the effects of hammer-blow and comparing the simultaneous records of deflexion taken on the two main girders. These results are shown in *Fig. 19* and have been plotted in relation to speed (miles per hour).

It is believed that the high values found in certain tests, at speeds between 35 and 40 miles per hour, were caused by some irregularities in the track which were not found elsewhere and may be regarded as exceptional.

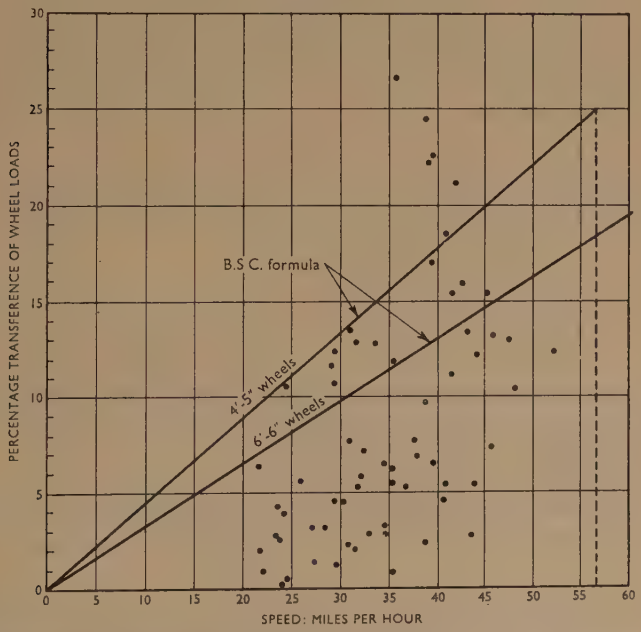
The two straight lines on the diagram represent the values given by the Bridge Stress Committee's formula applied to driving wheels of 4 feet 5 inches and 6 feet 6 inches diameters respectively.

It should be borne in mind that the experiments in the United Kingdom were made on railways of standard gauge and it might be expected that results on the wider Ceylon gauge would be less, in which case a reduced formula might be found appropriate, apart from the fact already mentioned

that in Ceylon the speeds related to a given engine frequency are considerably lower.

The diagram shows, however, that the British formula as it stands fairly covers the records made in Ceylon apart from the few instances already mentioned, and this tends to show that the lurching effects on the Ceylon railways are actually more severe than those believed to exist in the United Kingdom at corresponding speeds.

Fig. 19



Effects of Lurching, as observed, Compared with Bridge Stress Committee (1928) Formula

B.S.C. formula : Percentage transference of wheel loads $= \frac{n}{24} \times 100$

where n = no. of revolutions per second of driving wheels

In view of this information obtained during the tests, the Bridge Stress Committee's formula for lurching has been used in all the calculations of stresses made during the investigation in Ceylon, despite the fact that the speeds represented by the formula relating to the engines in Ceylon are so much less than those to be assumed in the United Kingdom.

Impact Percentages

In the tabulated calculations, impact has been expressed as a percentage of the static live load, including hammer-blow, lurching, and rail-joint effect. These percentages differ considerably for the two main types of bridge, that is, for deck bridges and through (or semi-through) bridges, for reasons already mentioned.

The allowances to be made for engines with no hammer-blow are, of course, very much smaller than those required for the ordinary type of two-cylinder steam engines such as are in use in Ceylon.

Fig. 20, Plate 1, has been prepared to show impact percentages as used in the calculations compared with values derived from certain other sources. These are the following :—

Bridge Stress Committee's Tables, deck spans at 4·5 r.p.s.
Bridge Stress Committee's Tables, through spans at 4·5 r.p.s.
British Standard, deck spans
British Standard, through spans
Government of India formula $I = \dfrac{65}{45 + L}$

As has already been mentioned, the effects of lurching in deck bridges as prescribed by the formulae in use and verified by experiment are important, and separate values have been plotted in Figs 21, Plate 1, for deck bridges and through bridges respectively.

It will be seen that there is a wide difference between the impact allowances recommended for the two types of bridge by the British Standard formulae and the same is true of values prescribed by the Bridge Stress Committee.

The Government of India formula is intended to apply to the design of all types of bridges and, for spans of 40 feet or more, appears to be borne out by the results of experiments in Ceylon on deck bridges but is excessive if applied to through bridges.

The British Standard formula appears to be on the high side for both types of bridge of more than 50 feet span. The Author has not attempted to devise any new formula as a result of the experiments in Ceylon and he would suggest that similar investigations in other countries would certainly yield useful information regarding the appropriateness of existing formulae to meet the conditions existing therein.

It has not been possible to detect, on the records taken, any clear indication of rail-joint effects and it is not believed that these in every case amount to a serious factor.

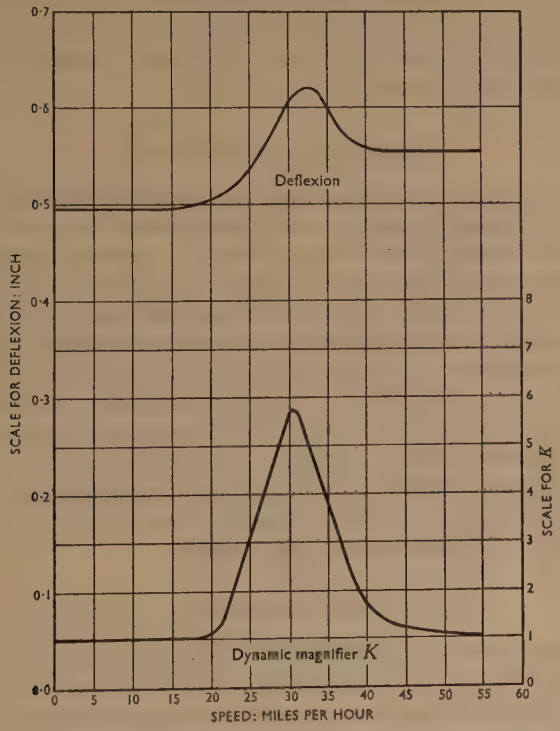
Typical Examples of Records Obtained

Figs 21, Plate 1, show the results obtained by tests with deflectometers on a span of 120 feet. The maximum oscillations occurred at a speed of 32½ miles per hour, the engine frequency being 3·46 cycles per second.

In *Fig. 22* the deflexion and the dynamic magnifier are plotted in relation to the speed.

Fig. 23, Plate 1, shows values of the dynamic magnifier recorded in tests

Fig. 22



Curves showing Maximum Deflexion and Dynamic Magnifier K at Different Speeds on 120-foot Span

on various spans plotted in relation to span. It will be seen that the values range from unity at 20 feet span to 8 at 160 feet span, and the maximum values follow a straight line.

In this diagram, for purposes of comparison, the values of K, the dynamic magnifier, given by the Bridge Stress Committee in Fig. 41, p. 87, of their report, have also been plotted.

It will be observed that tests on the Ceylon bridges yielded higher

values of K for all spans from 20 feet upwards. It appears that at 160 feet the values would agree.

Tabulation of the Results

Standard Stress Sheets were prepared for the tabulation of stresses in the girders of the bridges dealt with. They are filed in loose-leaf ledgers and form a permanent and convenient record for all future enquiries.

The detailed calculations for every bridge are contained in the original calculation books which are numbered and referred to on the Stress Sheets. These sheets are so designed that the stress in any girder under a future standard of loading can be obtained by a simple calculation, by using the factor given which indicates the stress in the girder produced by a load of 1 ton per lineal foot of track.

Results of Investigation

The investigation has shown that comparatively few bridges on the island are insufficiently strong to carry the engines now in use, including the new diesel-electric locomotives, and that the renewals or strengthenings immediately required are few in number. To render the lines in use suitable for engines of $16\frac{1}{2}$ B.S. units, if steam-driven, would involve considerable expenditure, even after the renewals now authorized and in hand have been completed. If the heavier engines were of the diesel-electric type, however, the expenditure would be materially reduced. The investigation, at the request of the Ceylon Government, did not extend to the light railways serving Batticaloa and Trincomalee, but the Chief Engineer, Way and Works, is continuing the work on lines similar to those adopted by the Author, with the intention of preparing a complete record of the strength of all the bridges in the island.

Acknowledgements

The Author wishes to acknowledge with thanks the help he received from Lt.-Col. Eagan, B.Sc., A.M.I.C.E., who took charge of the work during his absence on leave and was responsible, among other matters, for all the arrangements made to carry out the tests.

The work of Mr Bernard Bramall, B.Sc., A.M.I.C.E., especially in the stress calculations, was of great value, and in this he was ably assisted by Mr L. S. De Silva, B.Sc.(Eng.).

The Author also wishes to give special thanks to Mr E. C. Wijeyesekera, B.Sc.(Eng.), M.I.C.E., Chief Engineer, Way and Works, for placing at his disposal the resources of his department and for lending the services of his assistants. He is grateful also for the interest shown by the Chief Mechanical Engineer, Mr B. D. Rampala, B.Sc., A.M.I.Mech.E., who provided and equipped the testing engine.

The Paper is accompanied by 7 photographs and 18 sheets of diagrams, from which the Figures in the text, the half-tone page plates, and folding Plates 1 and 2 have been prepared.

APPENDIX I

Deflexion and Bending-Moment Diagrams for Bridges Loaded with Class B.1 Test Engine

The upper curve in Fig. 24, Plate 2, gives calculated values of $d \times I$ for any span when loaded by the testing engine, these values being those for each main girder of a single-line bridge, d denoting the deflexion in inches, and I the second moment of area of the girder section in inches[4].

If the deflexion caused by the engine when moving at slow speed is measured by a deflectometer, the effective value of I can be deduced by dividing the value on the graph by the measured deflexion.

The values of $d \times I$ have been calculated from the axle loads of the engine, which is assumed to be in the position in which maximum deflexion occurs. E is assumed to be 12,000 tons per square inch.

The remaining curves on the diagram give values of the product of I multiplied by the deflexion which would be caused by the hammer-blow of the axles acting as static forces and calculated in relation to the speed of the engine.

To determine the dynamic magnifier for a span under test, the deflexions of the girders are measured, first at crawling speed, and then at higher speeds reaching and, if possible, exceeding the critical speed.

If: D denotes the crawl deflexion;
$D + d$ „ the maximum deflexion recorded at speed;
A „ the value given by the graph headed "static load of engine" for the span in question;
and B „ the value given by the graph headed "normal hammer-blow" for the span and speed attained;

then, the dynamic magnifier K is given by the formula:

$$K = \frac{Ad}{BD}$$

The curves in Fig. 25, Plate 2, can be used to calculate values of K, the dynamic magnifier, from measurements of stress.

The upper curve, headed "static live load" gives values of the maximum bending moment caused by the testing engine moving at crawl speed over the span under test.

The other curves represent the bending moments which could be caused by the hammer-blows of the axles at various speeds acting as statically applied forces.

If: C denotes the bending moment (in inch-tons) given by the upper curve for the span in question;
F „ the stress in the girder, measured during a crawl run;
$F + f$ „ the stress measured at the speed which causes maximum stress;
and D „ the value given by the hammer-blow curve for the span at the speed attained;

then, the dynamic magnifier K is given by the formula:

$$K = \frac{C \times f}{D \times F}$$

APPENDIX II

Horizontal Rigidity of Bridges on Sharp Curves

The following indicates the calculations necessary to determine the fraction P of the transverse bending moment which is resisted by direct tension in the outer girder and compression in the inner one, based on measurements of the horizontal deflexions of the bridge during test runs.

The experimental value of the vertical moment of inertia of the girders is ascertained by the method described in Appendix I, based on the measured deflexion during a crawl run, making appropriate allowances for curvature and cant.

$$\frac{\text{Measured deflexion}}{\text{Calculated deflexion}} = \frac{I_c}{I_e}$$

where I_c denotes the calculated value of I and I_e the experimental value.

(*a*) *Horizontal Deflexion*

Centrifugal loading for the whole bridge is equal to :

$$\frac{0{\cdot}002025V^2}{R} \times W$$

where V denotes the speed in miles per hour, R the radius of curve in chains, and W the vertical load carried by one rail if the track were straight.

If I_h denotes the horizontal value of the second moment of area of the whole bridge section and D_h the horizontal deflexion, then :

$$I_h \times D_h = \frac{0{\cdot}002025V^2}{R} \times Y_i$$

where Y_i is the value obtained from Fig. 24, Plate 2, for the span tested.

(*b*) *Hammer-Blow Component*

If C denotes the cant, G the gauge, and H the vertical blow on the whole bridge at the speed attained in the test run, then the horizontal component of the hammer-blow is $H \times \frac{C}{G}$.

Y_2, the value of I times deflexion obtained from the graph of hammer-blow in Fig. 24, Plate 2, is based on half the track hammer-blow ; therefore, the value for the total horizontal function is :

$$I_h \times D_h = Y_2 \times \frac{2C}{G}$$

Total Transverse $I_h \times D_h$

The total transverse value of $I_h \times D_h$ is the sum of the two expressions in (*a*) and (*b*) above at the speed attained and the value is proportional to the square of the speed. It may be written that TV^2 = total value of $I_h \times D_h$ at speed V.

The theoretical effective moment of inertia of the bridge may be calculated :

(1) assuming flexure about the centre-line of the bridge, when

$$I_h = \text{gross area of girder} \times \frac{S^2}{2}$$

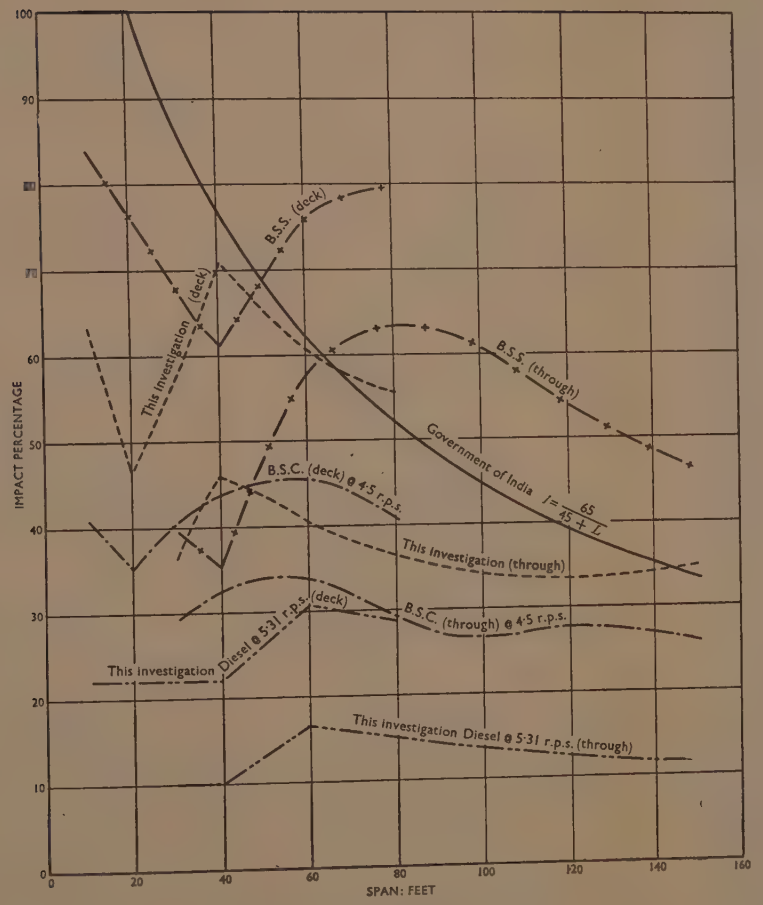
where S denotes the spacing of the girders ;

(2) assuming individual bending of each girder I_k = the sum of the effective horizontal moments of inertia of each girder.

If P denotes the fraction of the load which is resisted according to assumption

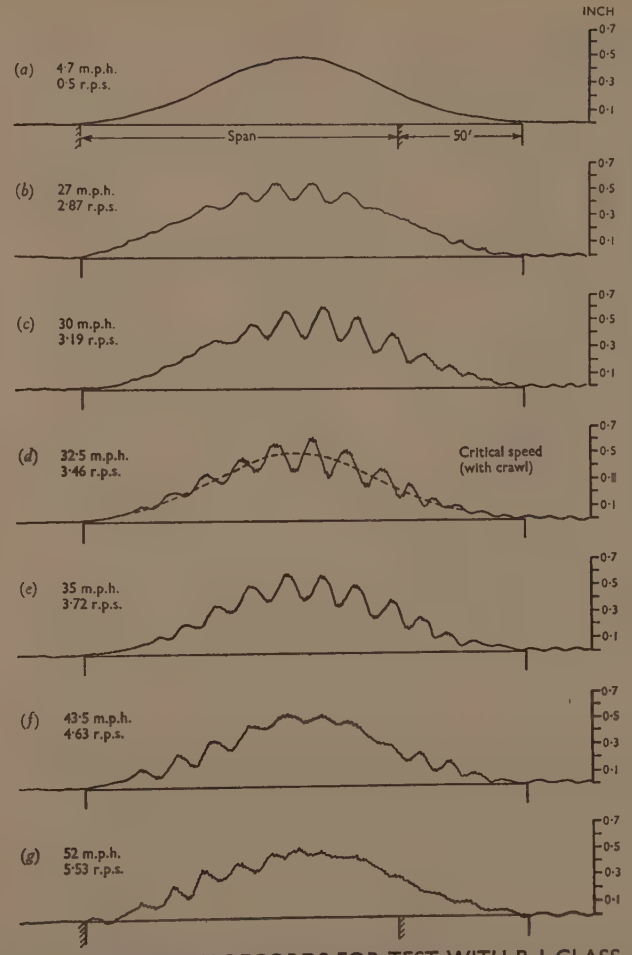
BRIDGE INVESTIGATION, CEYLON GOVERNMENT RAILWAYS, 1949–1951

FIG. 20


IMPACT PERCENTAGE
SPAN: FEET
B.S.S. (deck)
This investigation (deck)
B.S.S. (through)
Government of India $I = \frac{65}{45 + L}$
B.S.C. (deck) @ 4·5 r.p.s.
This investigation (through)
This investigation Diesel @ 5·31 r.p.s. (deck)
B.S.C. (through) @ 4·5 r.p.s.
This investigation Diesel @ 5·31 r.p.s. (through)

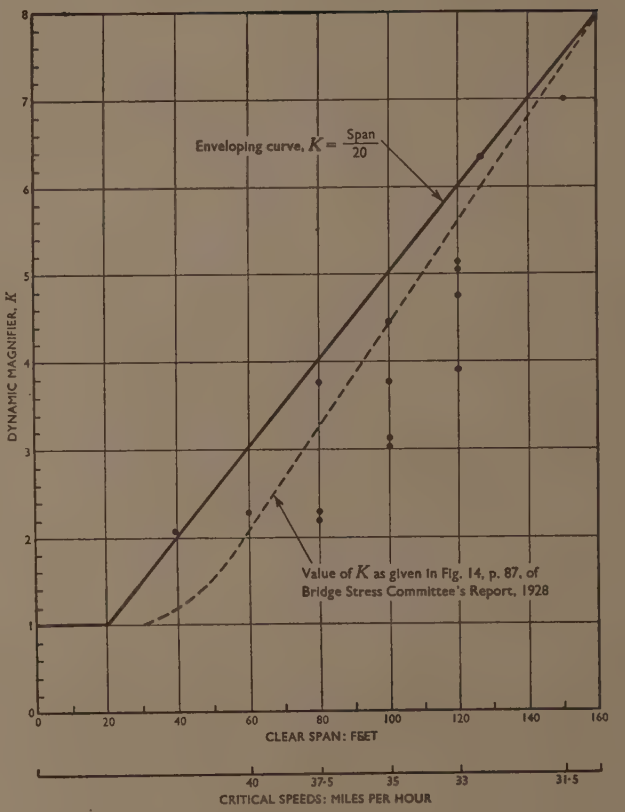
IMPACT PERCENTAGES
(British Standard, 1937, and Bridge Stress Committee Report, 1928)

FIGS 21



DEFLECTOGRAPH RECORDS FOR TEST WITH B.1 CLASS ENGINE ON 120-FOOT SPAN

FIG. 23


DYNAMIC MAGNIFIER, K
Enveloping curve, $K = \frac{\text{Span}}{20}$
Value of K as given in Fig. 14, p. 87, of Bridge Stress Committee's Report, 1928
CLEAR SPAN: FEET
CRITICAL SPEEDS: MILES PER HOUR

VALUES OF DYNAMIC MAGNIFIER K FOUND ON TESTS WITH VARIOUS SPANS

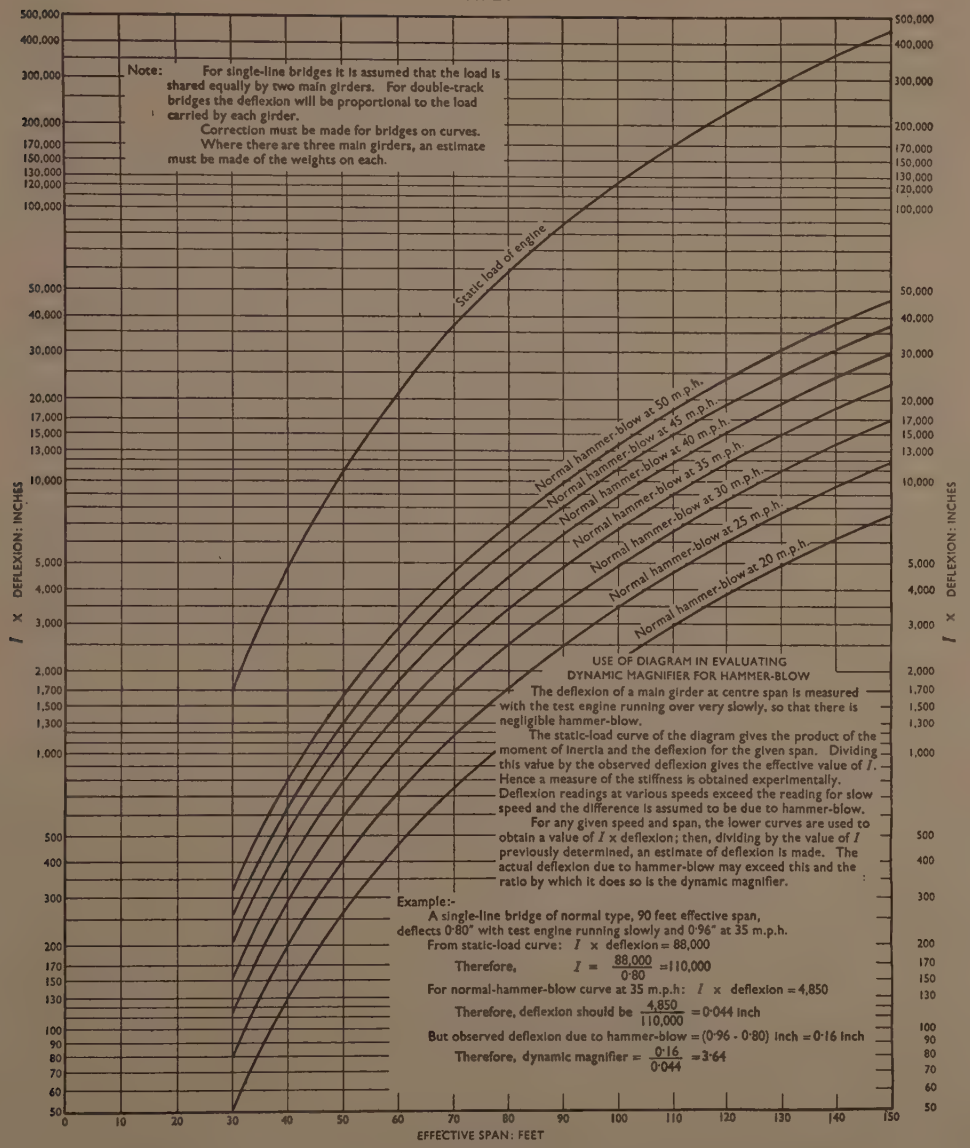
(For spans up to 50 feet, speed is the maximum—50 m.p.h. For spans of more than 50 feet, the critical speed is used.)

CONRAD GRIBBLE

WILLIAM CLOWES & SONS, LIMITED: LONDON

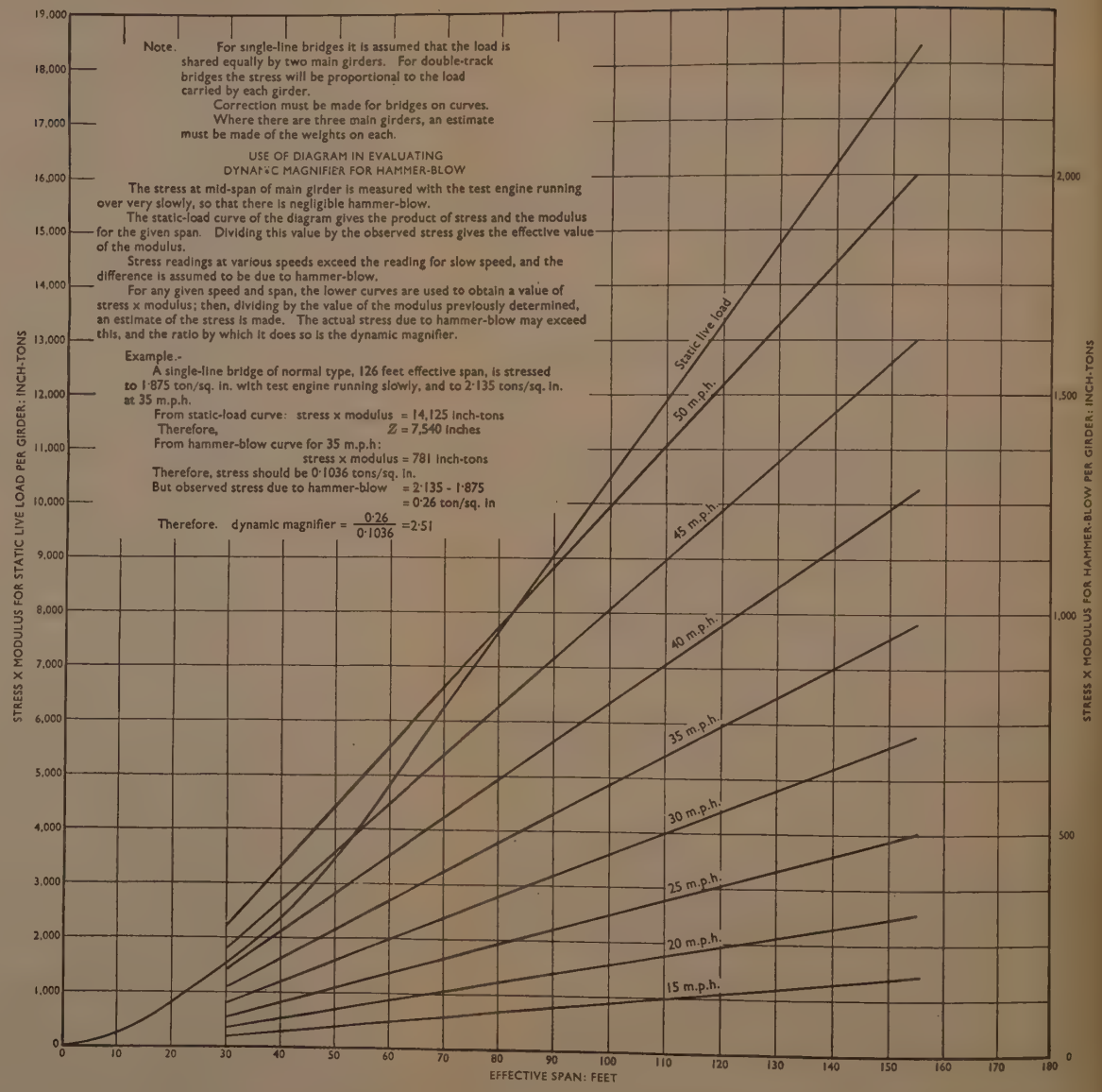
BRIDGE INVESTIGATION, CEYLON GOVERNMENT RAILWAYS, 1949–1951

FIG. 24


Note: For single-line bridges it is assumed that the load is shared equally by two main girders. For double-track bridges the deflexion will be proportional to the load carried by each girder.
Correction must be made for bridges on curves.
Where there are three main girders, an estimate must be made of the weights on each.
Static load of engine
Normal hammer-blow at 50 m.p.h.
Normal hammer-blow at 45 m.p.h.
Normal hammer-blow at 40 m.p.h.
Normal hammer-blow at 35 m.p.h.
Normal hammer-blow at 30 m.p.h.
Normal hammer-blow at 25 m.p.h.
Normal hammer-blow at 20 m.p.h.
I x DEFLEXION: INCHES
EFFECTIVE SPAN: FEET
USE OF DIAGRAM IN EVALUATING DYNAMIC MAGNIFIER FOR HAMMER-BLOW
The deflexion of a main girder at centre span is measured with the test engine running over very slowly, so that there is negligible hammer-blow.
The static-load curve of the diagram gives the product of the moment of inertia and the deflexion for the given span. Dividing this value by the observed deflexion gives the effective value of I. Hence a measure of the stiffness is obtained experimentally.
Deflexion readings at various speeds exceed the reading for slow speed and the difference is assumed to be due to hammer-blow.
For any given speed and span, the lower curves are used to obtain a value of I x deflexion; then, dividing by the value of I previously determined, an estimate of deflexion is made. The actual deflexion due to hammer-blow may exceed this and the ratio by which it does so is the dynamic magnifier.
Example:-
A single-line bridge of normal type, 90 feet effective span, deflects 0·80" with test engine running slowly and 0·96" at 35 m.p.h.
From static-load curve: I x deflexion = 88,000
Therefore, I = 88,000 / 0·80 = 110,000
For normal-hammer-blow curve at 35 m.p.h: I x deflexion = 4,850
Therefore, deflexion should be 4,850 / 110,000 = 0·044 inch
But observed deflexion due to hammer-blow = (0·96 - 0·80) inch = 0·16 inch
Therefore, dynamic magnifier = 0·16 / 0·044 = 3·64

DEFLEXION DIAGRAM FOR BRIDGES LOADED WITH CLASS B.1 TEST ENGINE

FIG. 25


Note. For single-line bridges it is assumed that the load is shared equally by two main girders. For double-track bridges the stress will be proportional to the load carried by each girder.
Correction must be made for bridges on curves.
Where there are three main girders, an estimate must be made of the weights on each.
USE OF DIAGRAM IN EVALUATING DYNAMIC MAGNIFIER FOR HAMMER-BLOW
The stress at mid-span of main girder is measured with the test engine running over very slowly, so that there is negligible hammer-blow.
The static-load curve of the diagram gives the product of stress and the modulus for the given span. Dividing this value by the observed stress gives the effective value of the modulus.
Stress readings at various speeds exceed the reading for slow speed, and the difference is assumed to be due to hammer-blow.
For any given speed and span, the lower curves are used to obtain a value of stress x modulus; then, dividing by the value of the modulus previously determined, an estimate of the stress is made. The actual stress due to hammer-blow may exceed this, and the ratio by which it does so is the dynamic magnifier.
Example.-
A single-line bridge of normal type, 126 feet effective span, is stressed to 1·875 ton/sq. in. with test engine running slowly, and to 2·135 tons/sq. in. at 35 m.p.h.
From static-load curve: stress x modulus = 14,125 inch-tons
Therefore, Z = 7,540 inches
From hammer-blow curve for 35 m.p.h:
stress x modulus = 781 inch-tons
Therefore, stress should be 0·1036 tons/sq. in.
But observed stress due to hammer-blow = 2·135 - 1·875
= 0·26 ton/sq. in
Therefore. dynamic magnifier = 0·26 / 0·1036 = 2·51
Static live load
50 m.p.h.
45 m.p.h.
40 m.p.h.
35 m.p.h.
30 m.p.h.
25 m.p.h.
20 m.p.h.
15 m.p.h.
STRESS X MODULUS FOR STATIC LIVE LOAD PER GIRDER: INCH-TONS
STRESS X MODULUS FOR HAMMER-BLOW PER GIRDER: INCH-TONS
EFFECTIVE SPAN: FEET

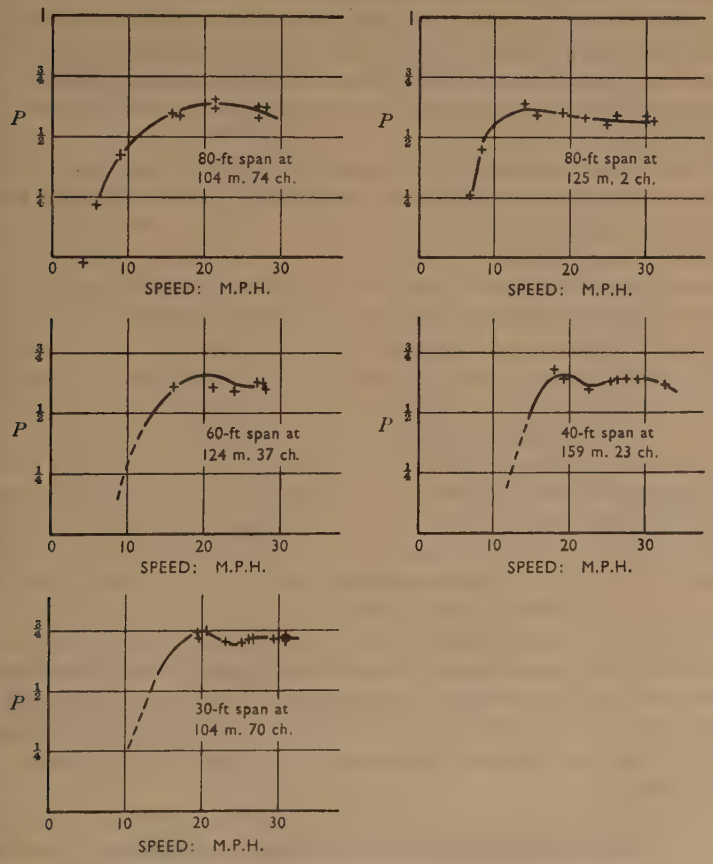
BENDING-MOMENT DIAGRAM FOR BRIDGES LOADED WITH CLASS B.1 TEST ENGINE

1) and $1-P$ the fraction resisted according to assumption (2), the theoretical trans-erse deflexion is :

$$D = TV^2\left(\frac{P}{I_h} + \frac{1-P}{I_k}\right)$$

If this is equated to the measured side deflexion, values of P can be obtained. uch values for various spans tested are shown in *Figs 26*. They indicate that at low

Figs 26



VALUES OF P FOR VARIOUS SPANS ON THE MAIN LINE

eeds where centrifugal force is small, very little of this force is resisted by the bridge ting as a rigid structure but when, at the higher speeds, the force increases, the bridge kes a much larger proportion of the load than the individual girders.

In these calculations the hammer-blow assumed is the "whole-engine" blow, hich is taken to be shared by the two girders.

Discussion

Mr W. E. Gelson first remarked upon the range of stress to which the wrought-iron specimens had been subjected in the 1936 fatigue tests. The range of $\pm 7{\cdot}5$ tons per square inch (see p. 118) was a figure which good wrought iron of the period might be expected to withstand.

He then referred to the calculation of horizontal deflexion in Appendix II. Since the bridges were of deck type, considerable difference in lateral deflexion due to centrifugal force was to be expected between top and bottom flanges. Might it be assumed that the term D_h denoted the mean deflexion ?

Were the end bearings on the shorter spans now of modern type, centralizing the loads on substructures, and did they provide for easy access for inspection and maintenance ?

Did the stress records on web members of lattice girders disclose a different mode of oscillation from the type found by the Bridge Stress Committee, and had the decision to use the same impact on webs as for booms been taken with the object of having an increased safety factor ?

The equipment which had been used in Ceylon for recording stress and deflexion had been of the same type as had been used in most of the work done by the Bridge Stress Committee, which had led to the report published in 1928. Although, no doubt, it had given good service in Ceylon, many additional data, valuable to bridge designers in search of increased economy and efficiency, could in Mr Gelson's opinion have been obtained if electronic recorders had been used instead of mechanical or optical recorders. The sort of instruments which he had in mind had been gradually developed and perfected for field use over the past 10 to 20 years. The American railways seemed to have been pioneers in developing small electro-magnetic extensometers, which had given valuable service in America and India in rail stress investigations from the late 1920s until the middle 1930s. They had then to some extent been superseded by resistance gauges, which had become commercially available shortly before the 1939–45 war.

The value of electronic strain-recording gear as a tool of research had been appreciated by that distinguished mechanical engineer, Sir William Stanier, and, as a happy result, two sets had been designed and built in the Derby locomotive shops in 1939. One of those sets had later been purchased from the L.M.S. Railway by the Indian Railways, on which it had since been used for important research work on track, on rolling stock, and for bridge impact investigations. The capacity of that particular recorder enabled up to eight synchronized records of stress or deflexion to be recorded simultaneously on one photographic roll film 6 inches wide.

From reports published in the Journals of the American Railroad Engineering Association in 1947 and 1949, it appeared that experimental

work on railway bridge impact had been resumed in America with the help of electronic gauge equipment. Had such equipment been used in the Ceylon tests, fewer line possessions might have been necessary and the field work would very likely have been simplified and hastened.

The Fereday and Cambridge instruments were not suitable in their standard form for use on members subjected to violent shocks, such as were produced by flats on tires and by poorly maintained rail joints. The rail-joint effect, being of a secondary order in the main-boom impact on anything but the shortest spans, was difficult to isolate in records from booms, and it could be studied conveniently only on records from gauges of low mass attached to floor members of large spans or to the main girders of very small spans. In cases where the provision of a rigidly fixed datum point for the measurement of horizontal deflexions was virtually impossible by reason of the height of the bridge or otherwise, a seismic mass attached to an ordinary deflectometer was usually effective at all speeds from about 20 miles per hour upwards.

He felt certain that if wider publicity had been given to that aspect of the work of railway engineers in Great Britain and in India, the Author would have taken steps to secure the co-operation of the British or the Indian railway research organizations.

Mr J. E. S. Bodger observed that the Paper showed the great need for specialized staff with access to high-class scientific instruments and testing machines, which were not always available to Colonial and other foreign railways. The Kelani bridge was an interesting case in point. Just before and immediately after the 1939–45 war, active consideration had been given to the reconstruction of that bridge on entirely new lines, which would have involved very heavy capital expenditure. That, he hoped, had been deferred for many years, and possibly for ever.

Had any investigation been made with regard to the substructure of that bridge ? He had not been in charge of bridges himself, but he knew that at one time that had given ground for anxiety. Also with regard to the Kelani bridge, it might be mentioned that the embarrassment to traffic of having a single-line section within 4 miles of the capital had been very great indeed, particularly during the war.

He had also been very interested to see the success of the centre girder which had been introduced on the up-country bridges by his one-time chief, the late Mr Cooper, a Member of the Institution, in a period when the Ceylon Railways had been very much hampered for money, during the world depression. That had been considered with regard to the 80-foot spans, or rather the boxing girders in those cases, but the war had held up that work.

He had no record of why Mr Cooper, who had then been in charge, had introduced " Chromador " steel, and he could find no record of when and how the maximum probable loading had come to be adopted. That had been in force when he (Mr Bodger) had gone to Ceylon in 1920.

Referring to one point which Mr Gelson had raised, Mr Bodger remarked that the rail joints over the Kelani bridge had been welded just before World War I, and they were standing up quite well. He had heard recently from the present Chief Engineer that the new Diesel locomotives were running very sweetly and having hardly any effect on the track at all. With regard to speeds, particularly up-country, he recalled that a test had once been carried out to discover whether a higher speed than 20 miles per hour would be practicable. After only one journey of the test train, the track had suffered considerable displacement, being in some sections 4 inches out of centre. Unless a heavy ballasting programme had been undertaken, it would not have paid to increase the speed, and it was impossible to have a regular speed of above 20 miles per hour. The many miles of 5-chain curves and gradients of 1 in 45 could set up considerable swinging.

Mr T. H. Hopkins referred to the Author's suggestion on p. 117 that the lower frequencies of bridges in Ceylon, compared with those in the United Kingdom, might be partly accounted for by their being of wrought iron instead of steel. Earlier in the Paper, on p. 107, he had stated that the ratio of depth to span was generally about 1 to 10 in lattice girders. Lattice girders were usually deeper than that, and it was from such deeper girders that the Bridge Stress Committee's frequencies had been obtained. Mr Hopkins thought that it should be remembered that, whereas the strength depended upon the first moment of area of the section, the deflexion (which was the controlling factor in frequency) depended upon the second moment of area, so that, given the same strength, a deeper girder would have a higher frequency; and it could usually be taken that the frequency, other things being equal, would depend upon the depth.

Opposite p. 160 of the Bridge Stress Committee's Report are listed a number of bridges of various spans, together with results of tests on frequency of oscillation. The Author had shown those frequencies by dots in *Fig. 9*. Mr Hopkins drew attention to the two bridges which had spans of 94 feet and 102 feet respectively and whose frequencies were very close to the line in *Fig. 9* showing the frequencies of bridges tested by the Author in Ceylon. It should be noted that those two bridges had depth/span ratios of 1:10 and 1:9 respectively (compared with the more usual proportion of 1:8. In the same list, said Mr Hopkins, there was also a bridge with a depth/span ratio of 1:13; that bridge had a high frequency which might at first sight appear to be exceptional, but it was significant that it was a skew bridge, crossing at an angle of 26 degrees, so that although the ratio seemed high, it was hardly a genuine case.

For the benefit of younger engineers he would say that working out the second moment of area did not necessarily give the correct formula, because there were so many other factors to be taken into account. In the plate girder it was the average second moment of area which was important, and the flooring was also significant. However, with practice, a very good degree of accuracy could be obtained by intelligent guessing.

The Author and those responsible in Ceylon were to be congratulated on using their old wrought-iron girders as strengthening, instead of new steel girders. The steel girders near the coast had all corroded badly.

A bridge over Borough High Street in London had been built in 1866, using wrought-iron main girders. In 1895, the bridge had been widened in steel. Mr Hopkins had been concerned with the painting of that bridge in 1905 and the steel girders, which had then been up about 10 years, were pitted all over, whereas the bloom had still been on the wrought-iron girders. By using the old wrought-iron girders in Ceylon, therefore, something much more permanent had been employed than would have been the case had new steel been used.

Mr P. S. A. Berridge observed that the practice in Ceylon seemed to be far behind that in India, where investigations such as the Author had described had been carried out by assistant bridge engineers in the course of their regular duties. All the staff had been familiar with the use of Fereday-Palmer recorders.

The Author had mentioned the limited resources for the repair and maintenance of delicate instruments in Ceylon. Mr Berridge was surprised that no use seemed to have been made of the pencil-and-card testing apparatus (*Fig. 27*), which was very simple to use. The pencil was clamped to the girder and the card was merely pinned with drawing-pins on to a piece of wood which was fixed to a post in the ground. That provided all that was necessary in measuring vertical deflexion and horizontal oscillation. It had been the rule on the old North-Western Railway in India that on every new or reconstructed bridge, card tests should be taken at all bearings and at the centre of at least one span.

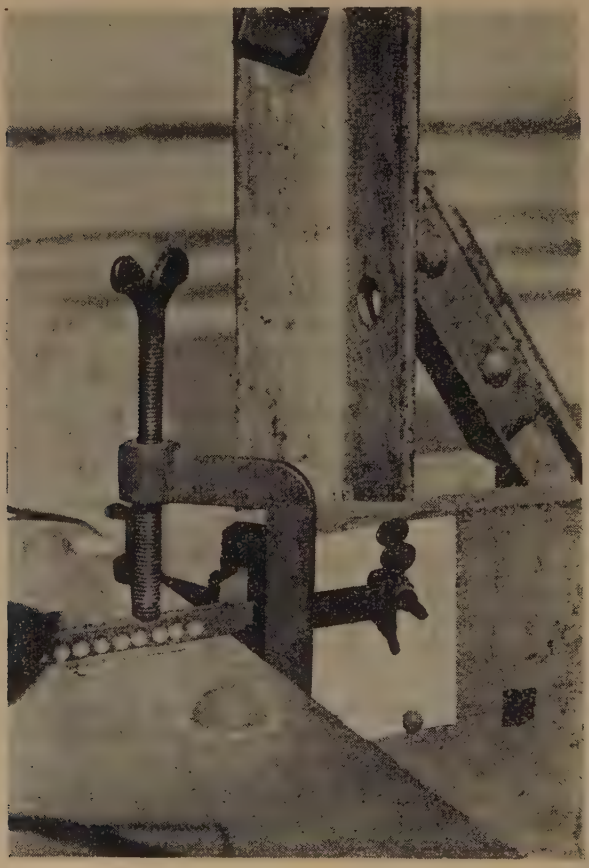
The Author had emphasized the need for providing lateral bracing on bridges on curves. Mr Berridge considered that lateral bracings were essential, even on straight track, if the girders of open deck spans were to be kept in proper alignment and the sway-bracing joints were to be kept from " working." Indeed, the 1941 Indian Steel Bridge Code called for all spans to be provided with a lateral bracing system extending from end to end, of sufficient strength to transmit to the bearings the racking forces, etc., as specified in the Bridge Rules. Spans of up to 65 feet were to be designed to carry wind and racking forces of 600 lb., treated as a moving load.

Clause 208 (*b*) of the Code of Practice for Simply Supported Steel Bridges, published jointly by the Institution of Civil Engineers and the Institution of Structural Engineers, required that allowance be made for a horizontal nosing force of 10 tons applied at rail level and at right angles to the track. Open deck spans with the bridge timbers hook-bolted to the girders had to be fitted with lateral bracing if those requirements were to be met. On through-type spans, top and bottom laterals should be fitted.

Mr Berridge showed a slide of a 200-foot span of an Indian bridge,

which clearly illustrated the horizontal bowing of the top chords of a through-type span of that length. The shallow sway bracing was quite ineffective when the chords of both girders bowed laterally in the same direction; in that case, the very flimsy sway bracing was quite useless

Fig. 27



Card-and-Pencil Apparatus for Recording Maximum Vertical and Horizontal Movements

to prevent the bowing from increasing. It had been necessary to impose a speed restriction of 5 miles per hour on that bridge for many years. One of the first actions taken by the North-Western Railway after partition, when it became Pakistan, had been to fit top laterals to the bridge and lift the speed restriction.

In 1945, some 105-foot semi-through pony-truss spans had been built

for the Nari bridge on the main line to Quetta. The Warren truss girders had six panels and a depth/span ratio of 1:6. An efficient system of bottom lateral bracing had been fitted and care had been taken to keep the bracings completely clear of the cross-girders and stringers, so as to avoid deformation stresses arising from interaction between the deck system and the main girders. There were 85 tons of mild steel in one of those spans. He mentioned that because in the reconstruction of the Bara bridge recently the new spans had a depth/length ratio of 1:5 and were of exactly the same length (105-foot centres) and had again six panels, but with the extra depth they had top lateral bracings, and the total weight of mild steel was only 71½ tons per span—a saving of 16 per cent compared with the pony-truss type of span used in the Nari bridge. Those two bridges were an indication of how the deeper trusses with top laterals could prove more economical, even though the ratio of depth to length was higher than would normally have been adopted in the past.

Mr J. McHardy Young referred to the full-size tests on the cross-girders (*Figs 12*). It was stated on p. 120 that "The smaller girder had failed by buckling of the web at the points of loading." Web buckling could occur only by shear or by bending, or by a combination of the two. At the point in question there was no shear stress, and the bending stress did not attain sufficient magnitude to cause failure either by buckling proper or by yielding of the material. On the other hand, it seemed more likely, from the data given in the Paper, that what had happened in that particular case was that the compression flange had failed by lateral buckling and that, of course, would cause torsion of the whole section, which might perhaps look like web buckling. More information on that test would be very useful.

On p. 122 it was stated that, "Neither girder showed any signs of overstress in the riveting although the stresses which theory would indicate as probable under the loads ultimately applied might", the Author had said, "be expected to produce a failure of the material in bearing and shear." Had those girders been riveted hot or cold? If the rivets had been hot there would be an inherent initial-shrinkage strain in each.

Would the Author give more information on the question of the horizontal deflexion referred to in Appendix II (*b*)? There seemed to be a certain element of contradiction in the Paper; on p. 129 was found the statement ". . . because the cross-girders, which are riveted to the tops of the main girders, provide some stiffness so long as the rivets remain tight." That seemed contradictory to the method which was put forward of taking for combined action the area multiplied by the square of the distance, and that particular method did not seem valid in the case of a deck girder with an open floor. It seemed more likely that what was taking place was, as the Author had said, that the two girders and the cross-girders were acting as a rigid frame of the Vierendeel type, but before it could be decided whether or not that was possible more details of the connexions would have

to be known. Did the Author believe that the connexions were in fact strong enough to develop the moments which might be expected in a girder of the Vierendeel type? If a comparison were made of the deflexions shown in *Figs 15* and *16*, the interaction was rather of the Vierendeel type than that which was suggested in the Paper, namely, as acting more as what might be called a plate girder.

Mr J. D. Watson asked whether the reduction, or even the total elimination, of hammer-blow from the steam locomotives had been considered, especially since there was one class of locomotive, the B.1 class, which apparently was ruling on a large number of bridges. From examination of the loading diagrams on p. 110, the B.1 locomotives were found to be just a little lighter than the Diesel locomotives if static loading only was considered. If the B.1 locomotives had been rebalanced so that their rotating parts were in balance without any reciprocating compensation, considerable operating benefits would have resulted and some of the bridge strengthening might have been avoided. As an example, possibly the restriction imposed on the Kelani bridge could have been modified to enable two steam locomotives to use the bridge at the same time.

Some engineers might wonder whether the re-balancing of locomotives in such a way was permissible. Mr Watson had been co-Author with Sir Harold Colam of a Paper[1] based on experimental work carried out on the Madras and Southern Mahratta Railway over a period of about 10 years. They had been able to convince themselves by those experiments that the outside cylinder steam locomotives in use on that railway could be re-balanced so as to eliminate hammer-blow, and such re-balanced locomotives ran for long periods in normal service without complaint or any observable adverse effects.

That Paper had provoked a good deal of discussion and correspondence, and there had been a particularly valuable communication from the South African Railways, who had apparently come to very similar conclusions. Mr Watson had been away from the Madras and Southern Mahratta Railway during the war, but upon his return in 1946 he had found that the policy of re-balancing the steam locomotives had been persevered with, and in fact even extended; in some instances complete classes of locomotives had been re-balanced. Nothing had come to light to show that those locomotives ran badly or were in any way more subject to nosing or fore-and-aft vibrations, nor were they subject to more wear, than normally balanced locomotives. It might be mentioned that in the experimental work they had had the co-operation of Mr R. Lean, the Chief Mechanical Engineer, who had begun by being a little doubtful about whether there was anything in the idea, but who, in the end, had supported it wholeheartedly.

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[1] Sir Harold Colam and J. D. Watson, "Hammer-Blow in Locomotives: can it not be abolished altogether?" J. Instn Civ. Engrs, vol. 17, p. 197 (Jan. 1942).

Although the balancing of locomotives had, no doubt, been outside the Author's terms of reference, the hammer-blow of the B.1 type of locomotive had been a matter which he had had to take into account and, if the policy of eliminating hammer-blow had been in force on the Ceylon Government Railways, no doubt he would have had to take that into account in his investigations and recommendations. From an economic point of view, Mr Watson suggested that it was cheaper to re-balance a class of locomotives than to strengthen a number of bridges, or even one large bridge.

Mr Arthur Dean said that, as he saw it, the problem which had faced the Author was " What can we do, and how can we find out in the shortest space of time whether we have to carry out major works or not ? " Whilst research with better instruments would have been useful and very interesting to many people, Mr Dean was not sure that it would have given as reasonable an answer so quickly as that which the Author had produced. One was apt to be led away at times by the information which could be obtained by modern instruments. The main job of railway engineers, however, was to keep the railways running—if possible, with the bridges which existed.

The problem had been one of applying the work of the Bridge Stress Committee, which had been primarily to define the basis on which new bridges should be designed in Great Britain, to the assessment of existing bridges, which was quite a different problem in some ways, as those who were concerned with it now appreciated to the full.

Referring to the fatigue tests, it was of interest to note that they had been carried out on specimens without holes in them. He had also been interested to see the significance of American work of fatigue testing of full-scale fabricated girders, where stresses at failure of 20 tons per square inch under static loading had been reduced to 4 or 5 tons per square inch after 2,000,000 cycles of repeated loading. He knew that the Author's view on that point differed from his own. The Author had made his recommendations on the basis of a permissible calculated stress of 8 tons per square inch in wrought iron. That was a figure on which British Railways had standardized for calculated stresses, knowing quite well that, in the practical case, the stresses were usually only about 80 per cent of calculated ones. If they tested a bridge, so that they knew with greater accuracy how the load was distributed between its several parts, and derived the stresses accordingly, they took the precaution of reducing the safe stress from 8 tons per square inch to 7. What safe stress had the Author used when applying the result of the tests carried out, which showed, as was usually found in Great Britain, that the actual stresses were lower than those derived by calculation ?

On p. 120, in relation to one of the problems, the Author had referred to a " moderate factor of safety." What did the Author consider to be a " moderate factor of safety " in a special case of that kind ? Mr Dean felt

that the basis adopted was sound in relation to the infrequency of traffic. It would be interesting to know how many other types of engine there were in addition to the one with the comparatively heavy hammer-blow, which was a light engine by British standards. In standardizing the locomotives for British Railways, there had been close co-operation between the civil and mechanical departments in settling the extent of counterbalance, and the heavy engines had much less hammer-blow than the lighter ones. It might be difficult to eliminate hammer-blow from the lighter engines in Ceylon without getting bad running. It was not so easy to eliminate it from those engines as it was from the larger and heavier engines used on bigger railways.

The record of the corrosion of wrought iron was very interesting and was in conformity with British experience.

Mr Dean had hoped that, from the results obtained, the Author might have put forward more views on the subject of lurching. Mr Dean agreed that it seemed scarcely right that lurching should be related to revolutions per second of the driving wheels ; he suggested that each locomotive had its own unique speed at which its lurching was maximum. He and others had found, when investigating derailments, that severe lurching could be caused by a synchronization of track irregularities, speed of locomotive, and boiler rolling frequency. Lurching could, in such cases, build up to such an extent that the load had been reduced to almost zero on one side of the locomotive and derailment had followed.

In dealing with similar problems in Great Britain to those with which the Author had dealt in Ceylon, it was necessary to adopt a different approach. The Author's approach had been correct in relation to a limited number of bridges of similar style and span and a limited number of locomotives. But the problem on bigger railways was quite different. On the bigger railways there was a wide range of locomotives and of types of bridge, and Mr Dean was beginning to think that, so far as British Railways were concerned, their future work should lie more in the direction of recording statistically the ranges of stress obtained in a bridge, rather than to seek to define and pinpoint all the precise elements which augmented the static loading on a bridge.

During a period of 48 hours, stresses under the passage of about 200 locomotives had been recorded on a particular bridge which carried the East Coast main line near Darlington. The maximum stresses had been recorded in a frequency curve, which followed the normal form of a " probability " curve. Only two peak stresses which approximated to the peak stresses calculated by adding the effects of static loads, hammer-blow, lurching, and rail-joint effects were recorded. On that part of the line, therefore, the peak occurred once in a hundred loadings. That made him wonder whether the traditional attitude adopted with regard to old bridges was sound, in view of the variations to which they were subjected, and the variable speeds and weights of engines and trains, and whether it was right

to adopt a peak safe stress to the complete aggregate stress caused by all those dynamic factors. It might well be that, in Great Britain, a bridge on which the speed was relatively low, but on which all the trains travelled at that speed, was getting the heaviest loading on a time basis—heavier, that was to say, than a bridge carrying trains running at widely differing speeds. Most of the loading on one of them was in the same range, but on the other, it reached a peak at infrequent intervals. British Railways were only at the beginning of their investigation into that problem, which was : ought they to define the safety of bridges against a safe stress related to the yield stress or against a life in relation to repeated loading ? As yet they did not know, but he believed that the Author took the view that it was the safe stress related to the yield point which should be used.

*** **Mr Bernard Bramall,** who had been associated with the Author on the work described, referred to the tests on bridges carrying curved track where it had been necessary to establish the manner in which lateral forces were resisted.

The series of 60-foot spans, which had been strengthened by the addition of a third main girder, presented an intriguing problem. From normal static considerations it had been impossible to apportion the loading between each of the main girders. Eighteen spans of that type had been involved, and of them, sixteen had had centre girders which were deeper, although slightly weaker, than the original outer girders.

A theoretical analysis had been developed, which was given in detail on p. 153. Tests had relied on deflexion rather than stress measurement, owing to the difficulty of discriminating between vertical and horizontal flexure when instruments could not be located on the central axis of the booms. The tests had given substantial support to theory, both at crawl and at speeds commensurate with marked centrifugal and impact loading.

Although the bridges were devoid of any system of horizontal bracing, the three main girders were connected by transverse vertical frames, which contributed materially in constraining them to act together in vertical deflexion. For normal design purposes sufficient accuracy would certainly follow from a simple assumption that at any cross-section, the absolute deflexion of the centre girder was equal to the mean absolute deflexions of the outside girders, always provided that adequate transverse diaphragms were used.

The mathematical treatment was applicable to any three-girder-type bridge with cross-girders that were continuous over the three points of support. In particular, some recent developments in double-track semi-through bridge practice on British Railways furnished an interesting example.

In the past, it had been customary to design cross-girders as simply supported on the centre main girder between the two tracks, although in

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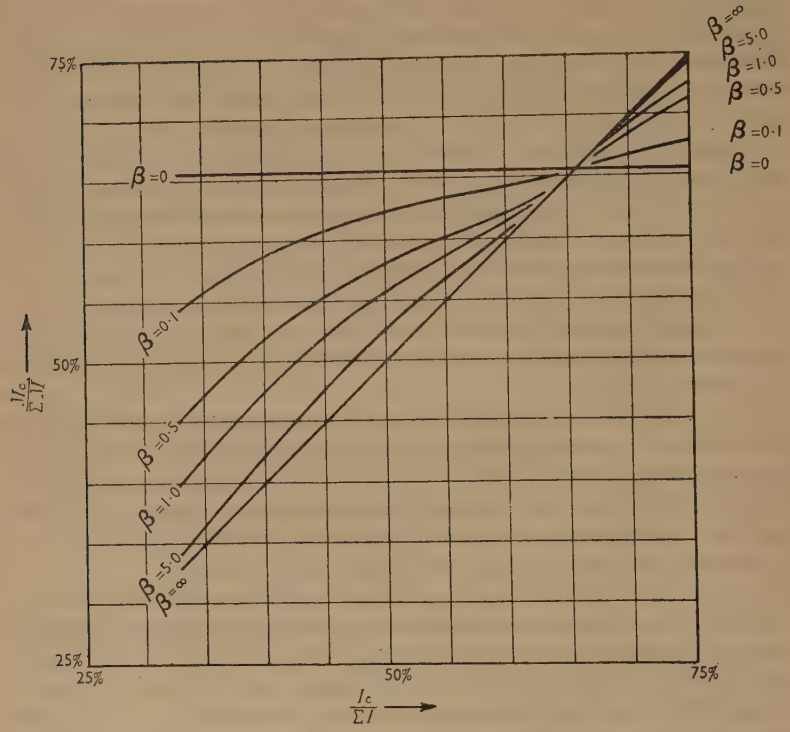
*** This contribution was submitted in writing.—Sec. I.C.E.

Fig. 28



Bending Moment in Centre of Centre Main Girder of 42-foot Span

Addendum to Figs 28 and 29

$$\frac{Ic}{\Sigma I} = \frac{\text{Moment of inertia of centre main girder}}{\text{Total moments of inertia of 3 girders}}$$

$\Sigma I = 75{,}000$ ins^2

With all main girders of equal depth, both graphs infer that total flange steel is held constant but cover a range of distribution of this steel between centre and outer main girders.

The range has been extended beyond practical limits to make clearer the effects.

$\frac{Mc}{\Sigma M}$ = proportion of total mid-span bending moment taken by centre girder.

$\frac{Fc}{\Sigma F}$ = proportion of total end shear taken by centre girder.

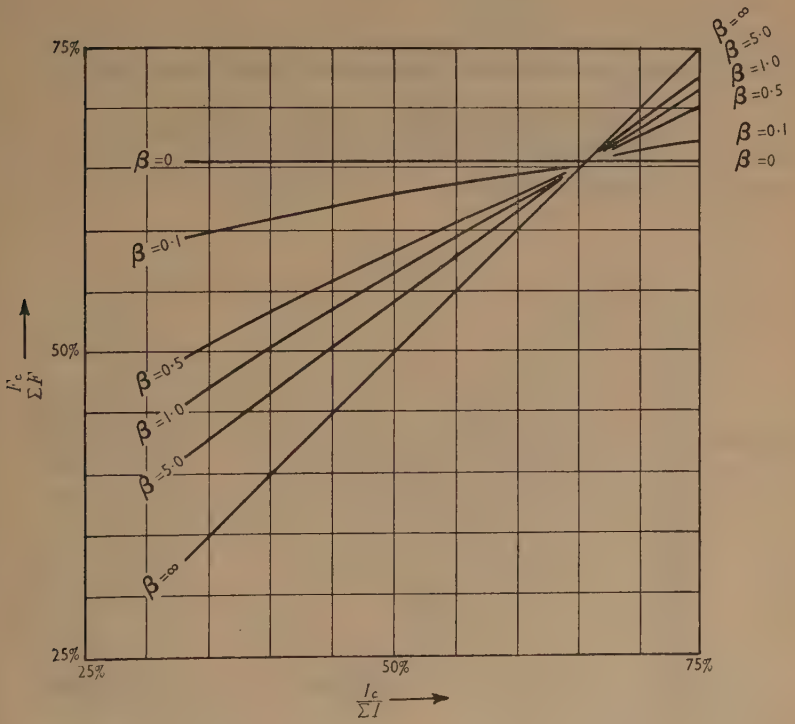
β represents the transverse stiffness of the decking

and is approximately
- 0·1 for steel troughing
- 0·5 for unit slab construction
- 1·0 for very rigid concrete slabs.

fact cleats had often been provided which denied flexing. Troubles had been experienced by such connexions working loose and by subsequent corrosion. In an effort to overcome that objectionable feature a number of bridges had been constructed, in which those connexions had been designed to transmit bending.[1]

Under those circumstances, and particularly when the connexions

Fig. 29



End Shear in Centre Main Girder of 42-foot Span

were opposite one another on each side of the centre main girder, a condition of transverse continuity existed. Advantage had been taken of that when designing the deck slabs by assuming partial continuity.[2] Clearly, however, that continuity was gained only at the expense of increasing the reactions on the centre support. Moreover, with one track

[1] P. S. A. Berridge, "The Design of Small Railway Underbridges with Special Reference to Erection and Maintenance under Traffic." Int. Assocn Br. & Struct. Engng, Zürich, 1952.

[2] "British Railways Through-Type Underline Bridges." *Rly. Gaz.*, vol. 97, p. 207 (22 Aug., 1952).

loaded, the adjacent side girder was penalized by greater loading than it was called upon to carry when both tracks were loaded.

Fig. 28 indicated the proportion of bending moment in the centre main girder with one or both tracks fully loaded.

As an example,

if

$$I_c = 35{,}000 \text{ ins}^4$$

$$\Sigma I = 75{,}000 \text{ ins}^4$$

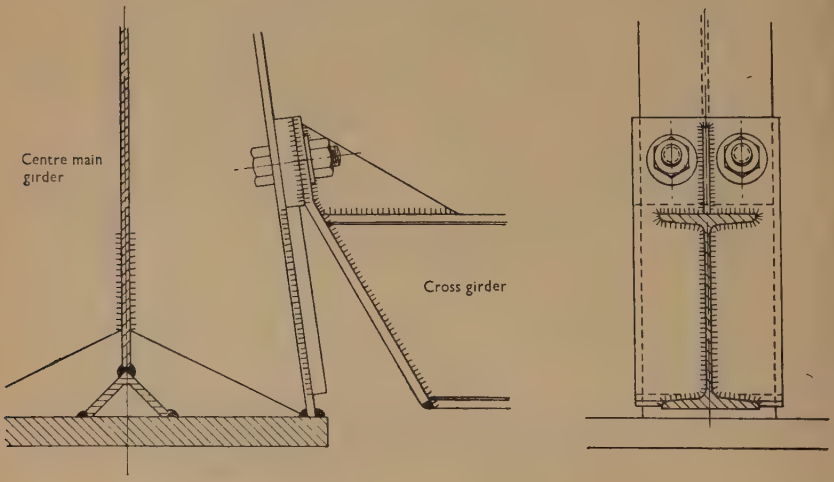
$$\beta = 0{\cdot}5$$

then the bending moment in the centre girder would be 56·3 per cent of the total in three girders, and if all main girders were of equal depth,

$$\frac{\text{Max. stress in centre girder}}{\text{Max. stress in side girders}} = \frac{56{\cdot}3}{43{\cdot}7} \times \frac{40{,}000}{35{,}000} = 1{\cdot}47$$

assuming both tracks to be loaded.

Figs 30



Either the centre girder was overstressed or the side girders were not economical in material.

Any further increase in the strength of the centre girder, accompanied by increase in its moment of inertia would result in yet higher loading, so that a vicious spiral prevailed up to the impracticable limit of 65·5 per cent.

Fig. 29 indicated in a similar way the proportion of end shear born by the centre main girder. Within the practical range those values were higher than for bending, but provision could be made to meet them without engendering further increment.

Uneconomic use of main girder steel was not adequately offset by advantages accruing from continuity in the decking.

Berridge preferred to stagger connexions either side of the centre girder, and insofar as that provided a limited flexibility, reducing the centre vertical reaction, the arrangement was to be commended.

End connexions of the cross-girders should, of course, be adequate to transmit any bending moment that their stiffness induced, but such moment should be regarded as an unwanted manifestation. Preferably the end plate should be regarded as a " shear plate," possessed of all the flexibility that could be contrived. *Figs 30* showed a possible solution for attaching cross-girders to the centre main girder, to meet the case where decking was placed after main girders were in final location. Shear load was transmitted through a top plate welded to the deck member, bearing upon a plate welded to the main girder stiffener, whilst the bolts were intended for positioning only.

ANALYSIS OF LOAD DISTRIBUTION IN THREE-GIRDER BRIDGES

(*a*) *Cross Stiffness*

The loading intensity on the centre main girder is expressible in the form :

$$r_c = \alpha w' - \beta d$$

where r_c denotes centre reaction per unit length.
w' ,, applied load on bridge per unit length.
d ,, deflexion of centre main girder relative to mean deflexion of the outer girders at the particular cross-section.
α is a constant.
β is a constant depending on stiffness of decking and cross-frames.

(*b*) *Main Girders*

Expressing deflexion at any point, distance x from one end of span of length L, thus :—

Deflexion of centre girder

$$= c_1 \sin \frac{\pi x}{L} + c_2 \sin \frac{2\pi x}{L} + c_3 \sin \frac{3\pi x}{L} + \ldots$$

Average deflexion of side girders

$$= a_1 \sin \frac{\pi x}{L} + a_2 \sin \frac{2\pi x}{L} + a_3 \sin \frac{3\pi x}{L} + \ldots$$

It follows that the bending moment at any point is :

for centre girder :

$$- \frac{EI_c\pi^2}{L^2}\left\{c_1 \sin \frac{\pi x}{L} + 2^2 c_2 \sin \frac{2\pi x}{L} + 3^2 \sin \frac{3\pi x}{L} + \ldots\right\}$$

for side girders (average) :

$$- \frac{EI_c\pi^2}{L^2}\left\{a_1 \sin \frac{\pi x}{L} + 2^2 a_2 \sin \frac{2\pi x}{L} + 3^2 \sin^3 \frac{3\pi x}{L} + \ldots\right\}$$

where I_c and I_a denoted the respective " equivalent " moments of inertia of centre and side main girders.

The shear force at any point is :

for centre girder :

$$-\frac{EI_c\pi^3}{L^3}\left\{c_1\cos\frac{\pi x}{L}+2^3c_2\cos\frac{2\pi x}{L}+3^3c_3\cos\frac{3\pi x}{L}+\ldots\right\}$$

for side girders (average) :

$$-\frac{EI_c\pi^3}{L^3}\left\{a_1\cos\frac{\pi x}{L}+2^3a_2\cos\frac{2\pi x}{L}+3^3a_3\cos\frac{3\pi x}{L}+\ldots\right\}$$

The loading intensity at any point may be obtained by differentiating the expressions for shear force. However, the following alternatives have much more rapidly diminishing terms :

for centre girder :

$$r_c=\alpha w'-\beta\left\{(c_1-a_1)\sin\frac{\pi x}{L}+(c_2-a_2)\sin\frac{2\pi x}{L}+(c_3-a_3)\sin\frac{3\pi x}{L}+\ldots\right\}$$

for side girders (average) :

$$r_a=\left(\frac{1-\alpha}{2}\right)w'+\frac{\beta}{2}\left\{(c_1-a_1)\sin\frac{\pi x}{L}+(c_2-a_2)\sin\frac{2\pi x}{L}+(c_3-a_3)\sin\frac{3\pi x}{L}+\ldots\right\}$$

The applied load, w' per unit length, may be variable or discontinuous. In the case when it is uniform and equal to w per unit length, the loading intensity on the centre girder at any point is :

$$\begin{aligned}&\frac{EI\pi^4}{L^4}\left\{c_1\sin\frac{\pi x}{L}+2^4c_2\sin\frac{2\pi x}{L}+3^4c_3\sin\frac{3\pi x}{L}+\ldots\right\}\\&\equiv\frac{4\alpha w}{\pi}\left\{\sin\frac{\pi x}{L}+\frac{1}{3}\sin\frac{3\pi x}{L}+\frac{1}{5}\sin\frac{5\pi x}{L}+\ldots\right\}\\&\qquad-\beta\left\{(c_1-a_1)\sin\frac{\pi x}{L}+(c_2-a_2)\sin\frac{2\pi x}{L}+(c_3-a_3)\sin\frac{3\pi x}{L}+\ldots\right\}\end{aligned}$$

Also, the total loading intensity of the three girders is :

$$\begin{aligned}&\frac{EI\pi^4}{L^4}\left\{(c_1+2a_1)\sin\frac{\pi x}{L}+2^4(c_2+2a_2)\sin\frac{2\pi x}{L}\right.\\&\qquad\left.+3^4(c_3+2a_3)\sin\frac{3\pi x}{L}+\ldots\right\}\\&\equiv\frac{4w}{\pi}\left\{\sin\frac{\pi x}{L}+\frac{1}{3}\sin\frac{3\pi x}{L}+\frac{1}{5}\sin\frac{5\pi x}{L}+\ldots\right\}\end{aligned}$$

From these two identities it is possible to determine all the unknown constants. Their values are as follows :

when n is odd :

$$c_n=\frac{\dfrac{A}{n}+\dfrac{c}{n^5}}{\left\{B_n{}^4+\dfrac{1}{2}\right\}\dfrac{I_c}{I_a}+1};\qquad c_n-a_n=\frac{\dfrac{A}{n}+\dfrac{A}{2}\dfrac{I_c}{I_a}-\dfrac{BC}{n}\dfrac{I_c}{I_a}}{\left\{B_n{}^4+\dfrac{1}{2}\right\}\dfrac{I_c}{I_a}+1}$$

when n is even :

$$c_n=0\,;\qquad c_n-a_n=0$$

where

$$A=\frac{4\alpha w}{\beta\pi}\,;\qquad B=\frac{EI_a\pi^4}{L^4\beta}\,;\qquad c=\frac{2wL^4}{EI_a\pi^5}$$

In the case when

$$w'=0\quad\text{for}\quad 0<x<\frac{L}{2}$$

$$w'=w\quad\text{for}\quad\frac{L}{2}<x<L$$

Then for n odd :

$$cn = \frac{\frac{1}{2}\left\{\frac{A}{n} + \frac{C}{n^5}\right\}}{\left\{B_n{}^4 + \frac{1}{2}\right\}\frac{I_c}{I_a} + 1} \; ; \; c_n - a_n = \frac{\frac{1}{2}\left\{\frac{A}{n} + \frac{A\,I_c}{2\,I_a} - \frac{BC\,I_c}{n\,I_a}\right\}}{\left\{B_n{}^4 + \frac{1}{2}\right\}\frac{I_c}{I_a} + 1}$$

for n divisible by 4 :

$$c_n = 0 \; ; \qquad c_n - a_n = 0$$

for n divisible by 2, but not by 4 :

$$c_n = \frac{-\left\{\frac{A}{n} + \frac{C}{n^5}\right\}}{\left\{B_n{}^4 + \frac{1}{2}\right\}\frac{I_c}{I_a} + 1} \; ; \; c_n - a_n = \frac{-\left\{\frac{A}{n} + \frac{A\,I_c}{2\,I_a} - \frac{BC\,I_c}{n\,I_a}\right\}}{\left\{B_n{}^4 + \frac{1}{2}\right\}\frac{I_c}{I_a} + 1}$$

With the full span loaded, the bending moment at centre of central girder is :

$$-\frac{EI_c\pi^2}{L^2}\{c_1 - 3^2c_2 + 5^2c_5 - 7^2c_7 + \ldots\}$$

The shear force at end of central girder ($x = 0$) is :

$$-\frac{EI_c\pi^3}{L^3}\{c_1 + 3^3c_3 + 5^3c_5 + 7^3c_7 + \ldots\}$$

or : $$L\left[-\frac{\alpha w}{2} + \frac{\beta}{\pi}\left\{(c_1 - a_1) - \frac{(c_3 - a_3)}{3} + \frac{(c_5 - a_5)}{5} - \ldots\right\}\right]$$

With half the span loaded, the shear force at the centre of the central girder is :

$$+\frac{EI_c\pi^3}{L^3}\{+2^3c_2 + 6^3c_6 + 10^3c_{10} + \ldots\}$$

The Author, in reply to Mr Gelson's comments on stress records on web members, said that there had unfortunately been only two stress recorders and a limited staff available, and, as he had already indicated, there had been many things which they would have liked to do but could not do. They had been unable to make anything like comprehensive tests on the web members of the lattice-girder bridges, but they had made a certain number, and had concluded that they would not be far wrong if they assumed the same impact factor throughout the truss, because in point of fact it was seldom, if ever, found that the crux of the matter was the stresses in the web members. He agreed that that assumption was only approximately accurate, but they regarded it as reasonable in the circumstances.

The term D_h in Appendix II should be taken to represent the mean horizontal deflexion.

The bearings of the shorter spans were ordinary flat bearings and did not centralize the load on a line bearing. They were, in the main, open to proper inspection.

The Author had naturally thought, before going to Ceylon, of the use of electronic and other short-gauge-length recorders and he had made inquiries at the Building Research Station and elsewhere with regard to the

latest developments in that type of apparatus. He had come to the conclusion, however, that it would not be wise to take out such apparatus to Colombo, where they could not be sure that there would be adequate facilities to keep the instruments in order. In Great Britain, where one had all the resources of the scientific instrument makers at one's command the position was quite different; but he had taken the view, and he believed that he had been right, that the simplest and most robust type of instrument was the proper one to use in Ceylon.

Mr Bodger had asked a question about the substructure of the Kelani bridge. The Author had raised that question with the Chief Engineer but had been informed that it was not within his (the Author's) terms of reference.

Mr Hopkins was quite right in what he said about the frequency of the bridges tested. What the Author had wished to indicate in *Fig. 9* was that the loaded frequency of the bridges in Ceylon had been on the low side. That was of some importance, because, when taken into account together with the smallness of the diameter of the driving wheels of the engines, it resulted in a lower critical speed over the bridges than would otherwise have been obtained.

Mr Berridge had referred to horizontal deflexions. The Author wondered whether he had read the paragraphs on p. 131 of the Paper, in which horizontal deflexions were discussed and the statement was made that "The tests on the Northern Line, however, did show in a marked degree the existence of horizontal vibrations at speeds higher than those which caused the greatest vertical movements," and the statement on the next page that "It has been recommended that all the lattice-girder spans should be reinforced with adequate sway bracing as soon as possible." The point which Mr Berridge had raised, therefore, had been fully dealt with.

The Author did not think that Mr Berridge could have observed in the Paper the reference to the Eagan deflectometer which they had made, which was rather better than the card and pencil device, in that it produced about a dozen records on each spool; each was a continuous record, showing the whole vibration, not merely a maximum.

Referring to Mr McHardy Young's remarks on the failure of the smaller girder under test it was possibly a compression failure on the flange which occurred first, but the web had been seriously buckled. The Author believed that the rivets in those girders had originally been driven hot rivets.

The Author had had in mind that the deck bridges, of both two-girder and three-girder types, acted somewhat as horizontal Vierendeel girders when subjected to centrifugal forces, and the extent to which they could do so was dependent upon the rigidity of the cross-girder connexions.

Mr Watson had referred to the elimination of hammer-blow in steam locomotives. The Author did not know whether the Chief Mechanical

Engineer, Colombo, had ever seriously considered that. The result of the inquiry which the Author had carried out had been that there was less strengthening work to do on the bridges in respect of the engines now in use than had been expected, and the decisions with regard to alterations to existing engines or the design of future heavier engines had been made after the Author had left the island. The C.M.E. had been unable to tell him whether any new and heavier locomotives would be Diesel or steam driven. He had said, however, that if they were steam driven they would probably be two-cylinder engines, and at the Author's request he had indicated a figure which should be assumed for the hammer-blow of those. It was realized that if a two-cylinder steam locomotive of $16\frac{3}{4}$ units was introduced a considerable number of bridge renewals would be required.

The Author appreciated the importance of Mr Dean's views with regard to safe stresses in bridges and also the work which had been done by British Railways in that respect. The problem in Ceylon had been quite different for the reasons which Mr Dean had stated. In Ceylon traffic was not frequent; on some of the branches there were only a very few trains per day, and speeds were comparatively low. Mr Dean had to face the problem of intense traffic with many different types of engines, and was approaching his problem in the right way.

On the question of whether there were other engines besides the B.1 type producing considerable hammer-blows, there were certain other types rather lighter than the B.1, with similar hammer-blows and there were a few engines which were considerably lighter and had a slightly heavier hammer-blow, but they were not important. It should be borne in mind that the bridges were subjected to maximum stress very infrequently and any possibility of fatigue appeared, therefore, to be remote.

The Author was grateful for Mr Bramall's contribution and felt that his analysis would be valuable to engineers designing bridges of the type in question.

Paper No. 5925

"A Method of Estimating Speed and Flow of Traffic from a Moving Vehicle" *

by

John Glen Wardrop, B.A., and George Charlesworth, B.Sc., Ph.D.

(*Ordered by the Council to be published with written discussion.*)†

SYNOPSIS

A method is given for measuring traffic speeds and flows by observations made from a moving vehicle. Observers in test cars driven in the traffic record their journey times, count opposing traffic, and keep a tally of overtaking and overtaken vehicles. From these observations, the mean speeds and numbers of vehicles passing along a street can be obtained for all classes of vehicle selected.

The practical application of the method is described and the errors occurring in practice are discussed. It is concluded that the method is efficient and practical, and is particularly suitable when a general picture of traffic conditions on a network of streets is required.

If the tally of overtaking and overtaken vehicles is not kept, a systematic error appears, but for many purposes this error is not serious.

Introduction

To make a survey of traffic conditions in an area or to investigate traffic improvement schemes, it is usually necessary to measure traffic speed and flow;‡ measurements are also often required of such quantities as the stopping time at intersections and the concentration of standing vehicles. In this Paper, a method of obtaining this information by making observations from a vehicle moving in the traffic is described. Yates[1] mentions the use of the moving-observer technique for counting pedestrians, and the "Traffic Engineering Handbook"[2] describes the use of a test vehicle for measuring traffic speeds. Although some experiments in the United States of America on speed-measurement by this means are described by Berry and Green[3] and the accuracy of the method discussed, the use of the

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† Correspondence on this Paper should be received at the Institution by the 15th June, 1954, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words—Sec. I.C.E.

‡ "Flow" is used here to mean the number of vehicles passing a specified point in a stated time.

[1] The references are given on p. 169.

method for measuring speed and flow simultaneously does not appear to have been given elsewhere.

Method

Consider a stream of vehicles moving along a section of road in one direction so that, on the average, the number passing through the section per unit time is constant. It is required to know what this number is and also what is the mean speed of the traffic.

The number of vehicles passing an observer per unit time depends on whether the observer is stationary or not. If he is moving in the opposite direction to the stream, the number will be higher than if he is standing still ; if he is moving with the stream, the number moving in the direction of the stream as counted by himself (that is, the number that overtake him minus the number he overtakes) will be less. In fact, the net rate at which vehicles pass the observer is proportional to his speed relative to that of the stream. If, therefore, the observer makes runs with and against the stream at known speeds and counts the flows relative to himself, then it is possible to evaluate the number of vehicles passing through the section per unit time and also the mean speed of the traffic. This is considered in more detail in Appendix I where the following results, in terms of journey times through the section, are derived.

Flow, $$q = \frac{(x + y)}{(t_a + t_w)}$$

Mean journey time, $$\bar{t} = t_w - \frac{y}{q}$$

where x denotes number of vehicles met in the section when travelling against the stream,

y ,, number of vehicles that overtake the observer minus the number that he overtakes in the section,

t_a ,, journey time of the observer when travelling against the stream, and

t_w ,, journey time of the observer when travelling with the stream.

Consequently, by counting x and y and measuring t_a and t_w, it is possible to evaluate q and $\bar{t}$. If n runs are made in each direction, the x, y, t_a, and t_w may be replaced by the average values for n runs.

It is seen that if $y = 0$, the journey time of the observer is equal to the mean journey time * of the traffic. The flows of different classes of traffic

* For $y = 0$, the "Traffic Engineering Handbook" [2] wrongly states that the speed of the observer would be the modal route speed. Overtakings depend on relative speed, and the numbers of overtaking and overtaken vehicles will be equal for a vehicle travelling at a speed such that the total of the deviations from that speed is zero—that is, for a vehicle travelling at the mean speed.

in a stream and their mean journey times can be obtained if x and y are counted for the separate classes.

In a one-way street no journeys against the stream would be possible, but by making two sets of runs with the stream, one at a low speed and the other at a higher speed, thereby obtaining two values of t_w and y, it would be possible to evaluate both $\bar{t}$ and q. To reduce errors, the two different speeds should be well separated from one another. This procedure could be adopted for traffic on a two-way road, but would be less efficient than the method already described.

Experimental Technique

The method has been applied to make surveys of traffic in London. Cars were driven in both directions along sections of road, observers in the cars recording the time-history of journeys in the sections, counting opposing traffic, and keeping a tally of overtaking and overtaken vehicles. The following procedure is based on that used in the surveys.

Equipment

A small, preferably even, number of test cars is required—usually two—each car carrying a driver and three observers. The cars should have good all-round vision and should be reasonably comfortable to minimize observer errors arising from fatigue.

One observer in the car counts opposing traffic, using electrical counters operated by push-buttons. The counters are mounted on a recording board, carried by another observer, on which is also mounted a watch, preferably graduated in hundredths of a minute to simplify subsequent analysis. Attached to the recording board is a journey log, prepared in advance, on which the second observer records counter totals and times at predetermined points *en route* together with the times of stopping and starting at intersections. The third observer records, on a separate log, the number of overtaking and overtaken vehicles and, if required, the number of parked vehicles.

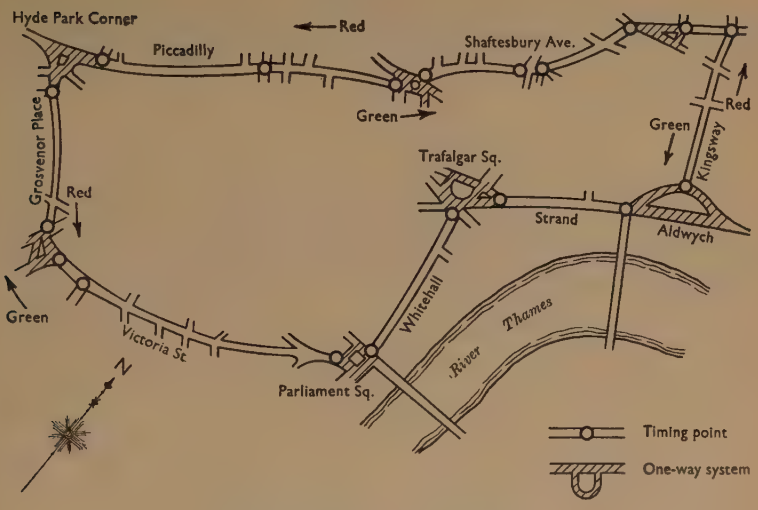
If a tally of overtakings is not kept, the driver of the test vehicle is instructed to overtake as many vehicles of a single class as overtake him. In this case only one class of vehicle or group of classes, which may include all types of vehicles, can be studied in detail at a time.

Routes

It is convenient, although not essential, for the routes studied to be closed circuits so that the test cars can start at various points and make complete journeys in either direction. A typical closed route is shown in *Fig. 1.* The two opposing directions are termed " red " direction and " green " direction.

Since conditions are not generally uniform along an entire route, the

Fig. 1



Map of Typical Route (Route Q)

route is divided into sections ; lengths of $\frac{1}{2}$ to 1 mile are convenient although a wide range of variation in section length can be accepted. The ends of sections should, so far as possible, include all the major intersections on the route so that large discontinuities in speed and flow do not occur inside the sections. *Fig. 1* illustrates a typical " breakdown " of a route (Route Q) into sections. Timing points are always placed at the entry and exit of any one-way system, but these systems are usually excluded from the analysis since they require special treatment, as has been mentioned.

Journey Log

Before experiments are started, a survey of the route is made and the journey log prepared ; an example for a portion of Route Q is given in *Fig. 2*. All intersections are identified by street names visible to the recorder as he travels along, the names being placed on the left or right of the column according to whether the street is on the left or right of the observer ; if the street runs right across the route, the name is placed in the middle of the column. Ends of sections are shown in large type and an indication of the form of control at all intersections is given. Cumulative counter totals and watch times are entered at all section ends and the time of stopping and starting at all intersections is recorded. If there is no delay, this is indicated by a tick in the start/pass column, and if there is a succession of stops and starts (for example, at a traffic light) these are entered separately on the log. The instant of passing an intersection is defined as the instant when the test car enters the intersection, for example, when it crosses the " stop " line at a traffic light.

In analysing the experimental data, blank log forms are used as analysis sheets, the differences in watch and counter readings being recorded on the sheets. Journey times are converted to speeds, using distances measured by a distance recorder attached to a car, or from Ordnance Survey maps.

Fig. 2

ROUTE Q DIRECTION RED SECTION I

Recorder R. F. Newby Observer J. R. Smith Car No. JXX 729 Driver C. Watters Date 1·10·52 Starting time 11:15 a.m.		Stop-watch times		Counter readings		
		Stop	Start / pass	1 Private	2 Commercial	3 Others
HYDE PARK CORNER — HALKIN ST (Exit to Grosvenor Place)	—		0·0	1864	7953	4211
Lower Grosvenor Place (Hobart Place)	L	0·74	0·87			
EBURY ST b	P	1·47	1·63 1·71	78	70	37
BUCKINGHAM PALACE RD	P	2·13	2·17 2·32			
VAUXHALL BRIDGE RD	P		2·75	83	71	40
Palace St. Ashley Place	L	3·29	3·43			
Buckingham Gate Artillery Row	L	4·01	4·26			
Broadway Strutton Ground	L		✓			
Tothill St. - Great Smith St.	L	5·39	5·46			
PARLIAMENT SQ. (Entry)	—		6·01	1901	94	63
PARLIAMENT SQ. (Exit to Whitehall)	P	6·45	6·88			
Horse Guard's Ave.	P		✓			
TRAFALGAR SQ. (Entry)	L	7·97	8·46 8·65			
Charing Cross Rd (St. Martins) (Straight on)	L		✓	18	8017	86
STRAND (Charing Cross Station) (Turn left)	P	10·06 10·46	10·22 10·66 10·76			
Villiers St. b	P		✓			
Exeter St. (Savoy)	P		✓			
WELLINGTON ST LANCASTER PLACE	PL		11·97	1951	8025	4327

b denotes street name visible in backward direction only

Vehicle Classification

Classes of vehicles should be clearly defined so that observers counting vehicles have no uncertainty about border-line cases. Vehicles that are held up temporarily (for example, at traffic lights) are included with moving vehicles, but it is sometimes difficult to distinguish such vehicles from those standing or waiting on the road, which are not included because they are not part of the flow.

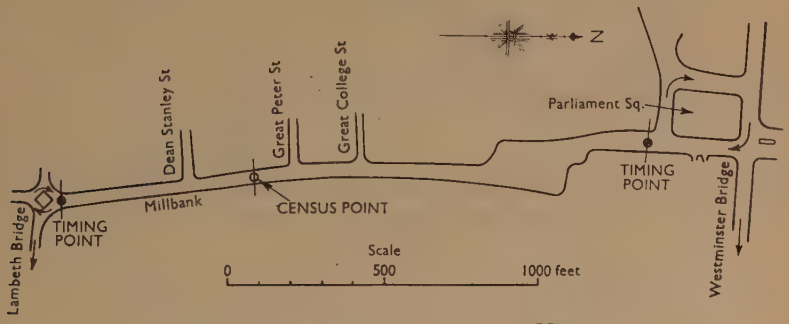
Number of Runs

It has been found that twelve to sixteen runs in each direction along the routes are usually sufficient to give reasonably consistent estimates of speed and flow.

Accuracy of Method

To check the accuracy of the method, the flows determined have been compared with those measured by a stationary observer at the road side. The test was made in Millbank between Parliament Square and Lambeth Bridge, a distance of about 500 yards; the route is shown in *Fig. 3*. A

Fig. 3

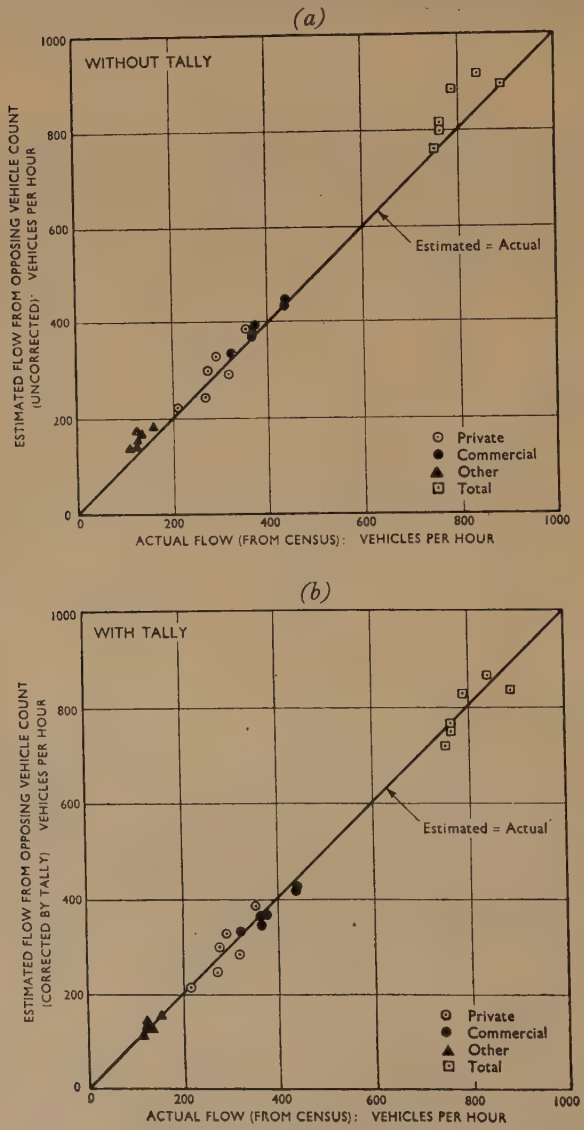


Route for Testing Accuracy of Method

series of runs in each direction was made using the method, and a census of vehicles was taken at the point shown. A correction varying from 0 to 5 per cent, based on sample observations of turning traffic at the three side streets, was applied to the census results to give the mean flow along the section. The values of the mean flow for each class of vehicle in each direction for three periods, as obtained by the census and by the opposing-vehicle method, are given in Table 1. These results are shown graphically in *Figs 4* (*a*) and (*b*), which show that for private and commercial vehicles the opposing-vehicle count, without the tally correction, gives unbiased estimates with a fairly small random error. The standard errors are 14 per cent for private and 3 per cent for commercial vehicles. The estimated values for other vehicles (buses, taxis, etc.) are, however, too high, the systematic error being + 25 per cent. This can easily be accounted for on the assumption that the average speed of this class is appreciably less than that of the test vehicle. Buses were included whether stationary or moving, and cruising taxis looking for fares are quite common in this area. The error could be reduced by counting buses only when they are moving.

This discrepancy in " other " vehicles gives rise to an error in the total

Figs 4



COMPARISON BETWEEN ESTIMATED FLOWS AND ACTUAL FLOWS ON MILLBANK

TABLE 1.—ACTUAL AND ESTIMATED FLOWS ON MILLBANK

		Flow : vehicles per hour											
		Private			Commercial			Other (buses, taxis, etc.)			Total		
	Period :—*	1	2	3	1	2	3	1	2	3	1	2	3
Southbound	Census	206	290	326	437	376	438	107	112	127	750	778	891
	Census (corrected for turning traffic)	**212**	**292**	**320**	**438**	**376**	**439**	**111**	**119**	**129**	**761**	**787**	**888**
	Opposing vehicles	220	327	292	444	389	437	134	169	165	798	885	894
	Opposing vehicles and tally	**213**	**320**	**284**	**426**	**367**	**422**	**112**	**141**	**127**	**751**	**828**	**833**
Northbound	Census	346	281	264	358	356	322	117	116	146	821	753	732
	Census (corrected for turning traffic)	**352**	**276**	**269**	**365**	**363**	**325**	**121**	**121**	**153**	**838**	**760**	**747**
	Opposing vehicles	384	301	244	378	370	334	152	139	182	914	810	760
	Opposing vehicles and tally	**379**	**297**	**239**	**359**	**343**	**330**	**128**	**125**	**152**	**866**	**765**	**721**

Date : 24 February, 1950.

* Times : Period 1 10.45 A.M.–12.15 P.M.
„ 2 1.50 P.M.– 2.50 P.M.
„ 3 3.40 P.M.– 4.40 P.M.

flow when the tally correction is not used. This is about 6 per cent in the present instance. However, when the tally is included, as in *Fig. 4 (b)*, these errors disappear, as they should. The remaining errors appear to be quite random and the standard errors are 8 per cent for " other " vehicles and 4 per cent for all vehicles.

This result shows that the full method can be used to give an unbiased estimate of flow. In the modified method referred to earlier, the driver is instructed to overtake as many vehicles of a single class as overtake him, so the tally for this class should be zero, and it is therefore not counted. However, experience shows that it is very difficult for a driver to follow this instruction in heavy traffic, and that in London a car driven normally, or at " about the speed of the surrounding vehicles," will in fact travel at a faster speed than the average for all vehicles. In a survey of Central London traffic in September–October 1949, the drivers were told to drive at about the speed of the traffic, but in addition the tally was taken. The results, given in Table 2, show that the error in the mean speed of all

TABLE 2.—ERRORS IN SPEED AND FLOW

(London Traffic Survey, September–October, 1949)

		Uncorrected	Corrected by tally	Error : per cent
Mean speed : m.p.h.	Private	11·4*	11·0	+ 4
	Commercial	11·4*	10·6	+ 8
	Other (buses, taxis, etc.)	11·4*	9·4	+21
	All vehicles	11·4	10·3	+11
Mean total flow : v.p.h.	Private	358	352	+ 2
	Commercial	521	501	+ 4
	Other (buses, taxis, etc.)	426	387	+10
	All vehicles	1,305	1,240	+ 5

* Assuming that all vehicles travel at the same speed.

vehicles was 11 per cent and that the error in assuming the speed of the test vehicle to be equal to that of "other" vehicles, mainly buses and taxis, was as high as 21 per cent. In each case the percentage error in flow was about half that of the speed, a result expected from the form of the expression for flow.

PRECISION OF THE METHOD

It has been shown that the full method gives an unbiased estimate of the flow. Random errors will occur, however, both from observers' errors and from random fluctuations in the flow. Observers' errors tend to increase with the magnitude of the flow being counted and with the number of classes of vehicle used, but experience shows that under normal conditions this is not a serious factor.

Errors arising from non-uniformity of flow always occur when a count over a sample period is used as an estimate of the mean flow throughout a longer period. This applies equally to a stationary census, when, for example, a few hourly counts during selected hours on one or two days may be used to estimate the total flow throughout a week or a month. The moving observer counts both the traffic that enters the section during his run and the traffic that was in the section when he started. The numbers are usually about equal, so that in terms of vehicles counted his observations are approximately equivalent to a stationary count over twice the single journey time or over the time for a complete return journey. It might be expected, therefore, that the random error of his estimate of flow, based on a number of sample runs, would be of the same order as that from a sample stationary count over twice the time spent travelling in the opposite direction to the flow being studied. In the test on Millbank, the

average return journey time was about 2 minutes (excluding turn-round time), and during a total period of 212 minutes covered by the stationary count, the time spent on return journeys was 96 minutes. The results obtained can therefore be compared with a stationary count covering 96 minutes.

The expected standard deviations of the differences between the flows derived from counts over a total of 212 minutes and over 96 minutes included in the total, are derived in Appendix II. The results are compared in Table 3 with the actual errors in the moving vehicle results.

TABLE 3

	Standard deviation of difference in flow between partial and full count : v.p.h.	Actual difference in flow between moving vehicle count and stationary count : v.p.h.	
		Southbound	Northbound
Private	10	− 5	+ 9
Commercial . .	13	−11	− 6
Other	7	+ 6	+ 4
All vehicles . .	20	−10	+ 7

It will be seen that the actual differences are of the same order as the expected standard deviations. This can be taken as evidence that an estimate of the mean flow based on moving vehicle counts will have about the same precision as that based on a sample census covering the same period.

MAN-HOURS REQUIRED

It is useful to compare the man-hours required for observations of flow and journey time by this method with the number required when using census-takers in conjunction with a test car. It has been shown that a series of runs, half in one direction and half in the other, will give estimates of classified flow, in each direction separately, of about the same precision as a stationary census covering the same period. To count classified flows in two directions, two census-takers would be required. On this basis, comparison between the new method and the more usual method can be made, as shown in Table 4.

This shows that if the speed of a single class of vehicles is required, the opposing-vehicle method without the tally is more economical in man-power than the usual methods, although the results are subject to a systematic error. In this case, the opposing-vehicle method including the tally is equal in terms of man-power to the usual method, but it has

TABLE 1.—RELATIVE OBSERVER-HOURS REQUIRED TO GIVE RESULTS OF EQUAL PRECISION

Method	Classified flow, single speed	Classified flow, classified speed (3 classes)
Census and test car .	3	5
Opposing vehicles without tally . .	2	–
Opposing vehicles with tally . . .	3	3

advantages in organization since census-takers would probably have to be transported from place to place.

If it is required to classify vehicle speeds as well as flows, the opposing-vehicle method with tally has a considerable advantage. In addition to saving 40 per cent of the observer-hours for a three-fold classification, only one third of the driver- and car-time is required. On the other hand, if the flow pattern only is required, the census method is more economical in man-hours.

ADVANTAGES OF THE METHOD

This method of measuring flow and journey time or speed has a number of advantages over more conventional methods :—

(i) It is economical in man-power if used without the tally for measuring classified flows and the average speed of a single class of traffic, and if used in full for measuring flows and average speeds of different classes of traffic. A small team of two to six observers, with one or two cars and drivers, can collect reliable traffic data on a considerable mileage of streets in 2 or 3 weeks.

(ii) It allows information on flow and on speed to be collected at the same time. This is particularly useful when the relation between the two is being studied.

(iii) It gives mean values of flow and of speed along the sections covered. Flow values measured at a point are liable to be affected by near-by intersections and may not be typical of a complete section of road. For some purposes, of course, spot values of flow and/or speed may be required, in which case this method is not suitable.

(iv) It allows additional information on stops at intersections, parked vehicles, causes of exceptional delays, etc., to be recorded if required.

Conclusions

(1) The method of measuring flow and speed of traffic from a moving vehicle described above is both efficient and practical. It is economical in man-power and is particularly suitable when a general picture of traffic conditions on a network of streets is required.

(2) When used in full, with a count of the tally of overtaken and overtaking vehicles, it gives classified flow results of equal accuracy to those obtained by a stationary census over the same period of time. It also allows classified speeds to be calculated.

(3) If the driver is asked to drive at about the speed of the traffic, a systematic error appears. Speeds are particularly affected, but in Central London the overall error in flow was only 5 per cent. Errors for private and commercial vehicles were 2 and 4 per cent respectively, whilst " other " vehicles, chiefly buses and taxis, showed a 10 per cent error. For many purposes systematic errors of this kind are not serious, since comparative figures are required. Experience with four surveys of Central London traffic suggests that comparisons based on this method are substantially correct.

Acknowledgements

The work described in this Paper was carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. The Paper is published by permission of the Director of Road Research.

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The Paper is accompanied by one photograph and three sheets of diagrams, from some of which the Figures in the text have been prepared, and by the following Appendices.

APPENDIX I

The Evaluation of the Flow and Mean Speed of a Stream of Traffic by a Moving Observer

Consider a stream of vehicles moving along a section of road, of length l, in such a way that the average number q passing through the section per unit time is constant. The stream can be regarded as consisting of flows q_1 moving with speed v_1, q_2 with speed v_2, etc.

Suppose an observer travels with the stream at speed v_w and against the stream at speed v_a. Then the flows relative to him of vehicles with flow q_1 and speed v_1 are $\frac{q_1(v_1-v_w)}{v_1}$ and $\frac{q_1(v_1+v_a)}{v_1}$ respectively.

If t_w denotes the journey time $\frac{l}{v_w}$ corresponding to speed v_w

t_a ,, the journey time $\frac{l}{v_a}$ corresponding to speed v_a

x_1 ,, the number of vehicles with speed v_1 met by the observer when travelling against the stream

y_1 ,, the number of vehicles with speed v_1 which overtake the observer when travelling with the stream (or minus the number he overtakes),

then

$$q_1 (t_a + t_1) = x_1$$

$$q_1 (t_w - t_1) = y_1$$

Similar results hold for all the other speeds, so that if

q denotes flow of all vehicles in the stream $= q_1 + q_2 + \ldots$

x ,, total number of vehicles met in the section when travelling against the stream

y ,, number of vehicles overtaking the observer minus the number he overtakes when travelling with the stream

$\bar{t}$,, mean journey time of all vehicles in the stream, that is,

$$\frac{(q_1 t_1 + q_2 t_2 + \ldots)}{q}$$

then summing over all speeds gives

$$q (t_a + \bar{t}) = x$$

$$q (t_w - \bar{t}) = y$$

or

$$q = \frac{(x + y)}{(t_a + t_w)}$$

$$\bar{t} = t_w - \frac{y}{q}$$

The mean speed is given by $\frac{l}{\bar{t}}$, which is the space-mean speed as defined by Wardrop.[4]

APPENDIX II

Derivation of Standard Deviation of Difference in Flow between Partial and Full Counts

Suppose that traffic is running with a variable flow, in such a way that the number of vehicles counted at a fixed point in each successive 2-minute period can be regarded

as a random sample from an infinite population with mean A and variance S^2. The period of 2 minutes has been chosen because the derivation is based on a sample made up of a number of 2-minute counts. It is not assumed here that traffic forms a random series, but merely that successive 2-minute counts are effectively uncorrelated, an assumption which is justified on the present data.

Let a series of N 2-minute counts be made, and let n of these be selected to form a sample period. Let Z denote the total number of vehicles (of a particular class) counted, and let z denote the number counted during the sample period. The following symbols are also defined :

$$m = N - n$$

$$w = Z - z$$

$$Q = \frac{30Z}{N} = \text{mean flow in } 2N \text{ minutes (vehicles per hour)}$$

$$q = \frac{30z}{n} = \text{mean flow in sample period of } 2n \text{ minutes (vehicles per hour).}$$

Then z and w are independent variables, with means nA and mA and variances nS^2 and mS^2. The quantity to be considered is $q - Q$, the difference between the flows over the sample period and the whole period. This is also a random variable, and has the value :

$$q - Q = 30\left(\frac{z}{n} - \frac{Z}{N}\right)$$

$$= \frac{30(zN - Zn)}{Nn}$$

$$= \frac{30(zm - wn)}{Nn}$$

Evidently the expectation of $(q - Q)$ is zero and its variance is :

$$S^2(q - Q) = \frac{30^2(nS^2m^2 + mS^2n^2)}{N^2n^2}$$

$$= \frac{30^2mS^2}{Nn}$$

In the present instance, $N = 106$ and $n = 48$, so that $S(q - Q) = 3{\cdot}2S$. Now the standard deviation of a 2-minute count can be estimated from the data obtained at Millbank, and the values found are as shown in Table 5.

TABLE 5

	Standard deviation of 2-minute count : vehicles
Private	3·1
Commercial . .	4·1
Other	2·1
All vehicles . . .	6·1

The values quoted in the Paper for $S(q-Q)$, that is, the standard deviation of the difference in flow between partial and full counts, are obtained from these figures.

Paper No. 5960

"Railway Architecture"

by

Frederick Francis Charles Curtis, Dr Ing., A.R.I.B.A.

(Ordered by the Council to be published with written discussion.) †

Introduction

It is probably no exaggeration to say that most people, particularly in Great Britain, do not readily associate the railways with architecture, although a magnificent start was made here in the pioneering days when some of the stations erected were among the finest buildings of their period.

Judging by references to it in novels, radio talks, newspaper articles, or in ordinary conversation, railway architecture seems to consist of grimy, ill-lit, draughty platforms with leaking roofs, dingy waiting-rooms, dilapidated refreshment bars, and millions of rather faded commercial posters. If the public show an interest in railways, it is usually in connexion with locomotives, their speed, and even their numbers, or with time-tables or engineering structures like tunnels or bridges.

A simple answer to the question as to how this came to be so is difficult to find. In the Author's view, the reasons for the low esteem in which railway architecture is held in Great Britain are first, the destructive and begriming effect of smoke in a damp atmosphere; secondly, the general meanness and mediocrity of railway stations carried out in the latter half of the 19th century, and the early years of the 20th century—characteristics which they share with many other buildings of the period; and thirdly, the disfigurement of many stations by later indiscriminate additions of lean-to sheds and commercial posters.

Unfortunately, at the present time, owing to shortages of capital, labour, and materials, little can be done to provide better railway buildings, and although there is every wish to improve the stations, both in their looks and in their usefulness, the public will not become aware of this until a good many improvements have been carried out.

Those who have an interest in this subject and are able to appreciate fine architecture when they see it, have, of course, been aware that there are a considerable number of railway stations in the British Isles which

† Correspondence on this Paper should be received at the Institution by the 15th June, 1954, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

have architectural distinction in a general sense, and at the same time are admirable examples of railway architecture in particular. It can also be said without exaggeration that the large number of stations and ancillary buildings erected for London Transport during the past 25 years are the most consistent attempt made anywhere in the world at developing an architecture worthy of a big transport undertaking. It is perhaps because of its undemonstrative excellence that this great achievement is little discussed or just taken for granted. Furthermore, it is an achievement confined to the London area and has no counterpart as yet on the main-line railways, particularly in the provinces.

It is hoped that if and when the latter can renew their buildings and execute a construction programme on a comparable scale, equally high standards will be reached. Leeds City Station is a fine pre-war example and, on a smaller scale, Luton, Doncaster, and Leamington Spa point the way.

Basic Considerations Influencing Design

What are the special characteristics of railway architecture, and the conditions and requirements which find expression in these special characteristics ?

To start at the root, consideration must first be given to the general background against which railway buildings stand. The most striking feature of a railway station, when compared with most other kinds of buildings, is the constant movement of persons and vehicles towards, through, and away from it. The observation of this fact has led some people to think that it does not matter what kind of stations are provided since everyone who uses them does so with the sole purpose of getting away from them as quickly as possible.

Upon entering any other kind of building, there is normally the feeling of withdrawing from the movement and noise outside, of finding shelter from the traffic of the highways, of coming to rest. Not so in a station, which is there to draw the traffic into it, and which, if well planned, leads this traffic as smoothly and expeditiously as possible in the direction in which it is intended to flow.

As a building, it is as static as any other, but much of it is a public thoroughfare and must, therefore, in its basic shape and arrangement, be expressive of movement. In saying this, the Author did not have in mind those rather childish attempts to streamline a building like an American car, but rather those features which are as natural to railway stations as limbs are to human beings : the drive into and out of the station forecourt, the canopies which give shelter to passengers alighting from cars at the station entrance, the footbridges across the line and the stairs leading up to them, the stairs down to subways and the subways themselves, the ramps up and down, the roofs over the platforms in their

relation to the length of trains, etc. At least some of these features are to be found at every railway station, and they do strike the keynote.

Furthermore, every railway station is a gateway through which those departing and arriving by trains have to pass. An early and also particularly fine expression of this function is the great Doric portico at Euston. On a very much smaller scale, but equally dignified, is the entry into the station at Rye, entirely appropriate in its simplicity to the setting of a small country town. A good modern example is the Amstel Station at Amsterdam, although this is, owing to the nature of the site and to the far more complex urban-traffic conditions in a modern city, no longer a simple gateway like Euston was when it was built.

Apart from being a gateway out of or into a town, a railway station is also a civic building of major importance. It is the reception room of a town where the visitor who has come by rail gets his first impression of the place he has reached. This aspect is one which, after the wonderful beginnings in about the middle of the 19th century, has been, on the whole, sorely neglected in Great Britain. The great hall in Euston, the lofty and nobly proportioned train halls at King's Cross and Paddington, before they were cut off from the streets outside by ancillary buildings, were worthy entrance halls leading into the metropolis. But who would think today of meeting his friends in a station concourse or restaurant unless this meeting is connected with a railway journey? Who would, after his arrival at Liverpool Street, and groping his way through the smoke-filled caverns under the Great Eastern Hotel, say to himself: "this is London, the heart of the Empire"?

There is nothing, alas, that would compare with Roma Termini, thronged every evening with promenaders, modern to a degree and yet carefully set into its surroundings, moulded around an ancient city wall which dates back 2,400 years. Nor is there a post-war booking hall as lofty, dignified, and truly urban as that in the Amstel Station at Amsterdam. It is very likely that most people in Great Britain have by now been mentally conditioned to the dingy cavern, so that it does not occur to them to wish for a fine reception room for their city. It is probable, however, that they would take very quickly to the idea if they could see at least one example.

Pride of achievement and the wish to display the Company's importance were often reasons for the magnificence of some of the early railway stations which, it should not be forgotten, had to accommodate at the time of their erection a very much smaller number of travellers than they do now. Today, it is hardly possible to justify display in the same sense, although there is always a certain publicity value in buildings which are worthy of a great transport undertaking and, in their high architectural quality, reflect the efficiency and comfort of the whole transport service. There is also much to be said, even from a purely practical point of view, for the use of good permanent materials, attractive in colour and texture,

as can be seen from the fact that some stations, now more than 100 years old, still serve their purpose admirably and do not require an unduly heavy expenditure on maintenance.

Examination of Specific Examples

In trying to describe the special characteristics of railway architecture, certain features have been mentioned which recur in almost every railway station throughout the world and which are expressive of the function of the building. First of all there are the great train halls such as Paddington and St Pancras, which find their counterparts in stations abroad in contemporary structures. They are now no longer typical for railway-station structures, but quite possibly complete electrification might bring them back. Difficulty of maintenance in a smoke-laden atmosphere is the main argument against them. They do, on the other hand, give far more complete shelter to the passengers and the staff. It is interesting that at Munich the bomb-destroyed station is being re-built with a lofty train hall, different in shape from the earlier examples, especially designed to allow for easy maintenance, and perhaps not quite so imaginative in form—but nevertheless a train hall with an overall roof. Needless to say, most of the traffic there is electrified, so that the smoke menace has been removed. A small example of a modern overall roof in Britain is White City Station of the London Transport Executive, where the lines are also entirely electrified. The already-mentioned Amstel Station is another modern example abroad, again on an electrified line. Compared with the train hall, the nowadays more prevalent umbrella roofing over the platforms only, lacks monumentality and, however well proportioned, can never be as effective in the architectural sense, especially in large stations with many platforms.

In small stations, with possibly only one island platform, the umbrella or canopy roof is, of course, the appropriate form. Moreover, owing to the small size of the building proper, it is in most cases the biggest architectural feature of the station. It is one of those structures which are often considered as engineering structures pure and simple, and outside the scope of the architect. Where this view has had an influence on the method of design—or its lack of method—the result is an awful conglomeration of unrelated shapes, roofs which do not fit on the buildings underneath, or buildings which somehow seem to miss fitting under the roof above. In this connexion, both in Paddington and St Pancras, and also in the modern examples of big train halls referred to earlier, engineers and architects have worked together in their design.

With regard to canopies over the entrances to stations, ramps and stairs, footbridges, etc., some examples will show how important they are in giving the building its special character. The façade of Roma Termini is dominated by the great canopy which projects from the booking hall

over the pavement and drive-in. Florence Station has a magnificent approach ramp and stairs, which give this very austere and unadorned station its distinction as a work of civic architecture. The early iron footbridges and recent reinforced-concrete ones are two different interpretations of the same design theme. Both can be equally graceful and fit for their purpose. The modern escalator is a further development of the station stair (although it is also found in department stores) and the Author personally likes the earlier examples with the arrangement of the light fittings on short bronze columns and of the stepped-up poster panels, which, by their repetition emphasize the length of stair, while at the same time counteracting the feeling of being sucked down a smooth pipe.

Some architects might accuse the Author of having given too much attention, in this Paper, to appearances and effects, as if the messianic puritanism of the functionalist, the planner, and the social economist had never tried to reform architects and turn them into useful research workers and citizens. On the other hand, there are engineers who still think that architects are a luxury when it comes to utilitarian building and are only required, if at all, to put the frills on the structures provided by engineers. These people will no doubt object to the inclusion, under the heading of "Architecture," of such items as footbridges and platform roofs.

The Lay-out and Design of a Typical Station

The Author's views on the scope of railway architecture can, perhaps, be best explained by a description of what he considers the right approach to the design of a railway station. In the first place—and no engineer or architect would disagree—an effort must be made to provide, at an economical cost, the accommodation which is required and to house it in a sound structure. That may be done quite well and yet the effort will still not produce a satisfactory station unless a great deal more is done besides.

The reader is asked to assume that an architect has given a good deal of thought to the design of a new station and has in his mind, therefore, a clear picture of the completed station after it has been put into use. Before committing himself to the preparation of contract drawings and working details, this architect makes a final inspection of the imagined building. He is determined to be critical, and take nothing for granted; he refuses to rely upon routine or stereotyped design where a fresh approach might result in improvements; he must not forget that the building is only one feature among many others in the railway scene as well as in its surrounding landscape or against its urban background.

His imaginary tour has an unfortunate start, because in this particular case, the station building is in a bay off a side street; the district is industrial and blackened by smoke; the surrounding buildings are mean and unattractive. He approaches *via* the main road, in the knowledge that the building itself cannot be seen very well, but must be easily found by

anyone who wants to travel from this station. He has arranged in his plan to erect a mast on a small traffic-island, in a position which can easily be seen from the main road. The sign on this mast can be illuminated at night. The mast itself is of a standard form in prestressed concrete and he has designed it to serve also as a platform lamp-post, either in an isolated position as in this case, or fitted into a standard trestle platform with standard concrete fencing. The sign on the mast directs him to the station building. Before reaching the building he lets a van turn into a small yard on the left to collect parcels from the parcels office. A taxi overtakes him and stops by the entrance canopy. The fence on the left, which hides the parcels yard from view, consists of poster panels, which are continued on the wall under the entrance canopy. Through the glazed entrance doors he has a straight view through the booking hall on to the platform. The building is constructed of engineering bricks with flat concrete roofs. No paint is used on any external surfaces, except on the underside of the canopy, which is painted in a light vellum colour to give reflected light, and on the metal windows, which are painted white. He thinks the posters look tidy and add a welcome touch of colour in this somewhat sombre neighbourhood. The doors are in teak, with anodized aluminium handles and the metal windows are set into teak frames.

He notices that the straight vista to the platform is emphasized by the light fittings, arranged in a straight line from the entrance canopy, through the booking hall, on to the platform, and the grey-black quarry-tile floor, which he can see extending through the whole building.

He enters the heated booking hall, which serves also as a general waiting space. Under the big window on his right is a teak seat, cantilevered from the wall to facilitate the cleaning of the floor underneath. To his left, the booking hall widens out and allows passengers to book tickets, hand in or withdraw parcels, or study the timetables without standing in the way of those who pass straight through from the street to the platform and *vice versa*.

He notices that the walls of the booking hall are tiled right up to the ceiling in a very light buff colour which blends with the grey-black quarry-tiles on the floor and the anodized aluminium frames around the ticket windows. Only the ceiling is painted (white) and the rest of the booking hall can be kept clean by washing down. The ceiling is of sound-absorbent material to deaden the sound of traffic over the tiled floor.

A cove at the junction of walls and floor can be easily swept and prevents the accumulation of dirt in the corners. All corners of the wall tiling are also rounded.

Facing the ticket windows he sees on the left of the glazed screen, which divides the booking hall from the booking office, the counter of the parcels office. On the right is a recess, corresponding in size to the opening of the parcels office, in which the timetables are displayed. They are illuminated by a strip light in the soffit.

Leaving the booking hall at the platform end through another pair of teak doors, he passes the ticket collector and reaches the platform. The concrete roof of the station projects over part of the platform, along the whole length of the building, and at one end is carried a short distance beyond the building. The underside of the platform roofing is a completely flat surface, interrupted only by the light fittings and the suspended direction signs. The positions of the posters have been agreed with the Publicity Department and chosen to fit in with the spacing and rhythm of the windows on the platform wall of the building. The station name-plates are placed at regular intervals, in positions where they are well lit and so spaced that one at least can be seen from any compartment in a train drawn-up at the station.

The portion of the platform roofing which projects beyond one end of the building, besides giving additional cover to waiting passengers, also gives shelter to persons leaving the station by additional gates leading directly on to the street. These gates are provided for rush traffic to and from nearby sports grounds and a race course.

From the platform, passengers can enter the lavatories and the ladies' waiting room. There is also access, from a lobby, to the Station Master's Office, the booking and parcels offices, a staff lavatory, and the basement heating chamber. The only other rooms in this small station, also accessible from the platform are a station store and the porters' messroom. The lavatories are, like the booking hall, tiled throughout.

In this particular case, the old platform building on the other platform is not yet due for renewal and will, at this stage, be merely redecorated. The old footbridge is also retained.

Standing under the new platform roof, with the main station building behind him, the designer now sees a typical northern industrial townscape, a mixture of utilitarian structures, of railway tracks, signal gantries, telegraph poles and wires, a road overbridge in the distance and, beyond the opposite platform, factories and smoking chimneys. The little building through which he has just walked is a very small object in this weird and dirty prospect of industrial ugliness. The tidy lines of the track and the platforms, the trains themselves, and the simplicity of the clean outlined, tidily arranged station building will be all that creates civilized order in the surrounding chaos. People may shudder at the idea of using smooth and hard engineering bricks for the (load-bearing) walls, but their use ensures that the rain will wash the walls clean and that hard wear will not scar the surfaces. The proportions of the building, the light-blue colour of the direction signs, the warm colour of the teak window-surrounds and doors, the added colours of the posters, placed in positions where such splashes of colour will be a visual relief and not a nuisance, are the designer's means of achieving the desired result. But in looking around, he is aware how many hideous objects there are everywhere and how much they could all be improved if proper thought were given to their design.

So far as these objects belong to the railways, work is already in hand to improve their appearance. The lamp posts, the seats, the lighting fittings have already been mentioned. The designer's team, which includes both railway engineers and architects, has also tackled the design of what might be termed "trackside furniture," which includes such items as mile posts, boundary posts, gradient posts, lengthmen's huts, signal gantries, etc.

He continues his tour of inspection and looks behind the scenes, where the station staff expect the best that can be done for them. This is a small station and should be worked by a small staff. At times when there is little traffic, it should be possible for one man to look after the booking and parcels offices. This man must, therefore, be able to supervise both places from wherever he happens to be, and should not have to walk long distances between the two offices. The Station Master needs to be in a central position from where he can reach, without delay, any part of the station. Where work has to be done, the lighting must be adequate, both during daylight hours and at night. Lighting fittings have accordingly been arranged in the soffit over each ticket window and over the parcels counter.

A small station as described above is about to be built in Scotland.

General Factors to be Considered

With reference to station design in a more general way, it is the Author's opinion that, apart from being practically planned and soundly constructed, a station building, or rather, a group of station buildings—for there is seldom only one building—must fulfil the following requirements:

It must be a good neighbour and blend in with its surroundings. It must have the modesty or boldness of appearance which is appropriate to its relative importance, it must look equally well from all points of view, inside and outside, it must be so designed that its main elements appear as such and are not overlaid by secondary ones, that approach, entry, and exit are obvious and unhindered, that the circulation is self-evident from the spatial arrangements, so that users need not rely entirely on direction signs in order to find their way. The building and all its details must be properly related in its scale to that of human beings, and it should be felt by (and give pleasure to) the users of the building, whether they be the travelling public or the railway staff, that imagination has been applied throughout its design.

The clerk in the booking office should be enabled to work in comfort, and at the same time this should be the kind of comfort which assists him in *doing his work*. The kind of comfort referred to here differs from that sought by the passenger who goes into a refreshment room. What the latter would wish to see might distract the other from his work.

Using a refreshment room are those who delight in watching the crowds moving through a station, especially if they themselves sit on a comfortable chair in a sheltered and quiet corner. There are others who seek a niche in which they can read their paper over a glass of beer and see nothing of what goes on in the station. Others again are nervous of missing their trains (or they are railway enthusiasts) and they would wish to have a view of the trains entering and leaving the station. Most people dislike stations cluttered up untidily with commercial posters, but most of them are also curious enough to look at them. People do not mind their presence in stations if they are tidily arranged and do not hinder the passenger in trying to find train indicators, direction signs, or timetables.

The seating provided on platforms and in waiting rooms should be comfortable, pleasant to look at, and pleasant to touch. Every detail, every fitting or piece of equipment should be well designed in itself and in relation to its background, and should stand out from, or merge into, the background as appropriate. The colour scheme of the painted surfaces should blend in with the background of the surroundings of the building, with the colour of the trains, with that of the materials used in the structure, and it should be so related to the background and lettering on direction signs that the passenger can easily see and read these signs. It is also quite a legitimate requirement that a railway station should have a publicity function of drawing the public to the railway service offered to them.

This is by no means a complete list of requirements which have to be fulfilled in a railway station building. No architect would get very far in his endeavours to produce a good building if he tackled all these requirements separately. Like the composer of a symphony who, when noting down the first bars of the work, must have already and simultaneously in his mind the sound and sequence of the whole of the music which follows, the really creative architect must be capable of visualizing all the parts of the building simultaneously and from all angles. He must be equally able to think of the shapes and plan arrangements in terms of structure as of the structural elements in terms of architectural appearance.

Standardization of Unit Construction

Since there are various basic elements which are to be found in each station building, it has exercised the mind of many people as to whether there could not be a considerable degree of standardization in station design. Not only the equipment and lay-out, but also the working methods, are similar in many stations throughout the country, and on the face of it, a standard unit construction seems to offer great advantages. Components could be bulk-ordered at a cheap price; prefabrication of standard units which can be easily assembled on the site would reduce erection time and avoid undue disturbance of traffic; building maintenance

and repairs would be simplified, it would be more easily possible to develop a standard " livery " of the stations within one railway system ; mass-production would make it economically justifiable to develop standard components which are adapted to special railway requirements.

The London, Midland and Scottish Railway Company developed a system of unit construction, shortly before nationalization, which was based on a planning module of 3 feet 4 inches. It was designed to allow for erection by the Company's own labour. At the time, it was anticipated that there would be a considerable programme of station renewals over a long period, and that bulk ordering of components in large quantities would make the application of such a system of unit construction an economical proposition. The system adopted allowed for considerable variations in lay-out and appearance, to suit local conditions, and could be applied to both large and small stations. It had, particularly in the finish of the platform walls with vitreous-enamelled steel plates, many advantages over most concrete structures—which get dirty and cannot be cleaned. However, owing to the restrictions in capital expenditure, only two prototype stations have been erected, one at Stonebridge Park on the Euston/Watford line and one at Marsh Lane, Liverpool. Up to now, the station-building programme of British Railways has been so restricted that it would have been uneconomical to continue using the system.

The disadvantages of a standardized unit construction, apart from its dependence for economical reasons on a large and steady building programme, are in the Author's view :

(1) The possibility of delays if one of the component parts is not readily available, owing to temporary shortages of materials.
(2) The length of time needed in developing a practical system. This might render the system obsolete before full production has been started.
(3) The difficulty of adapting an established system of unit construction to progressively changing methods of fabrication, and that of effecting desirable improvements without considerable cost in modifying the dies and the formwork as well as the machine tools required in manufacture.
(4) The reduced scope for initiative and inventiveness, so far as the technical staff is concerned, and the resulting restricted use made of skill available among the staff.

The Netherlands Railways have developed a system of prefabrication in reinforced concrete which is the best of its kind seen by the Author. It consists of a post-and-lintel structure with roof and wall panels and does not differ fundamentally from many others. Where it differs is in the standard of finishes, which is very high indeed. Concrete is not used as a substitute for other materials, as is so often the case, but as a material

which combines strength with beauty. The accuracy of casting is superb and great care has been taken in obtaining a permanent finish in colour and texture. Large surfaces of concrete usually look drab and uninteresting. By the use of pierced or patterned panels and of different coloured aggregates, and by contrasting surface treatments by sandblasting and polishing, this usual drabness has been avoided. The stations built in this system vary considerably in lay-out and detail; the building programme was, however, big enough to require the production of components in fairly large quantities, and thus to make it economically possible to undertake many tests over a long period.

Generally speaking, prefabrication has its biggest scope in small line-side buildings, such as lengthmen's huts. These are often situated in remote districts, and it is a great advantage in such cases if the complete hut can be prefabricated and pre-assembled in the works and then placed by crane on a prepared site. Small storage sheds, fogmen's huts, shunters' cabins are other examples.

There is, however, a strong case for the use of easily and quickly assembled prefabricated buildings in places where the station has to be kept open for traffic during building operations, and where it is essential to cut down the time needed for erection and to avoid the disturbances normally caused on the site if traditional methods of building are employed.

Conclusion

Apart from stations, of which there are more than 6,000, there are, of course, many other kinds of buildings in use or required by British Railways, some of which—for instance, signal boxes—are as special to the railways as the stations themselves. However, by far the greatest part of the building programme since the last war consisted of workshops and repair sheds, storage buildings, offices, canteens and welfare buildings, hostels, and houses. In connexion with the last-named, it might be interesting to mention that British Railways own 51,000 houses. The size of the building programme, so far as it relates to improved staff amenities and welfare, can be gathered from the fact that an expenditure of £1,500,000 was authorized in 1951 for welfare projects.

Other fields in which there is scope for architectural work include the design of coach interiors and the passenger accommodation on the railways' ships.

It is hoped that this Paper gives an idea of the wide scope there is in a railway undertaking for architectural design, and how useful a part architects can play in keeping the railway service up to date.

Paper No. 5886

"Construction of Reinforced-Concrete Ferry-Berth Pontoons in the Port of Sydney"

by

Basil Robert Monckton, B.E., A.M.I.C.E.

(*Ordered by the Council to be published in abstract form.*) †

History

Reinforced-concrete pontoons for ferry berthing were used in Sydney as early as 1913–14. The first one constructed, weighing 600 tons and measuring 110 feet long, 67 feet 7 inches wide, and 7 feet 9 inches deep, was believed, when launched, to be the largest vessel of its kind in the world.

The construction of further pontoons was recommended in 1945, and a dry-dock inspection of existing pontoons showed practically no deterioration after they had been in the water for 16 years. It was considered that with modern methods, similar pontoons could be expected to have a life of at least 40 years.

Hull Details and Construction

Fig. 1 gives details of the supporting girders and formwork for the lower slab, and also shows the various stages for the removal of the forms to enable the Army bridging pontoons to be inserted beneath for lifting purposes.

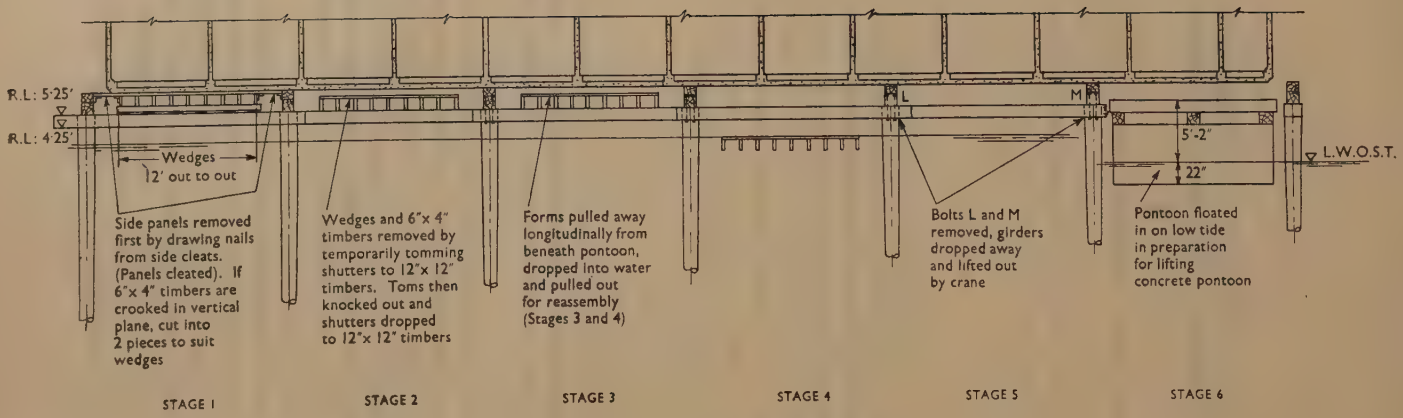
Fig. 2 is a typical cross-section of the vessel and shows details of the reinforcement used. The pontoon is a cellular vessel containing forty-eight watertight compartments and is 101 feet 6 inches long, 49 feet 4 inches wide at the bow end, 59 feet 6 inches wide at the aft end, and 8 feet deep. The outside "skin" walls are 5 inches thick and bulkhead walls are 4 inches thick. The top and bottom (5 inches) are strengthened by beams at the junction of the walls and bottom.

Construction was in three pours and the construction joints are indicated on the cross-section in *Fig. 2*. No trouble has been encountered because of leakage through the joints while the vessel has been in service.

Ordinary round mild-steel reinforcement was used and, where possible, continuously welded in a flash-butt-welding machine. The minimum

† The full MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

Fig. 1


R.L: 5·25'
R.L: 4·25'
Wedges
12' out to out
Side panels removed first by drawing nails from side cleats. (Panels cleated). If 6"x 4" timbers are crooked in vertical plane, cut into 2 pieces to suit wedges
Wedges and 6"x 4" timbers removed by temporarily tomming shutters to 12"x 12" timbers. Toms then knocked out and shutters dropped to 12"x 12" timbers
Forms pulled away longitudinally from beneath pontoon, dropped into water and pulled out for reassembly (Stages 3 and 4)
L
M
Bolts L and M removed, girders dropped away and lifted out by crane
5'-2"
22"
L.W.O.S.T.
Pontoon floated in on low tide in preparation for lifting concrete pontoon
STAGE 1
STAGE 2
STAGE 3
STAGE 4
STAGE 5
STAGE 6

PONTOON STAGING AND FORMWORK REMOVAL

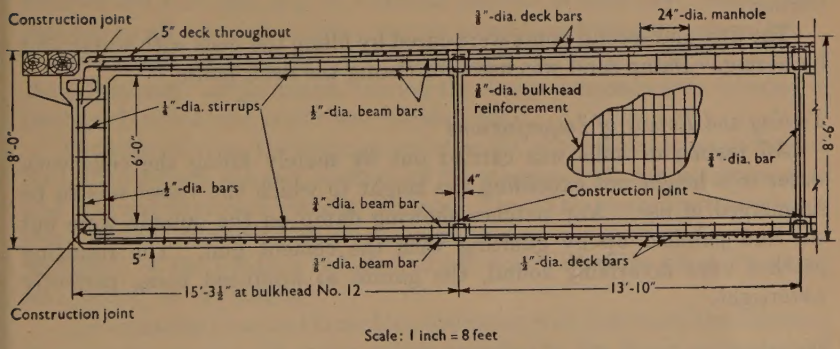
cover of concrete over the reinforcing steel on all external walls and slabs except the top deck was $1\frac{3}{4}$ inch.

Concrete

Natural aggregates were used in all concrete mixes, round river gravel of $\frac{3}{4}$ inch maximum size being used in the bottom slab and deck, and $\frac{1}{2}$ inch crushed river gravel in the walls. River sand with a small proportion of $\frac{3}{16}$ inch crushed blue-metal screenings constituted the fine aggregate.

To provide a concrete which would be highly resistant to the action of sea-water and also as workable as possible, it was considered that a mix containing at least 8 bags per cubic yard (94-lb. bags) would be necessary. A leaner mix was used for the deck, which does not come in

Fig. 2



Scale: 1 inch = 8 feet

TYPICAL TRANSVERSE SECTION

contact with sea-water. Low-heat cement was also used because of its sulphate-resisting qualities. Table 3 of the MS. shows the aggregate grading for a bottom pour; Table 4 shows the final dry mix; and Table 5 shows the results of strength tests made on cylinders 12 inches high by 6 inches diameter.

All aggregates were batched by volume. Two plants were used—a floating plant of 1 cubic yard and a shore plant of $\frac{1}{2}$ cubic yard capacity.

Slump was maintained at 3 inches for the bottom and top slabs and between 5 inches and 6 inches for the walls. All concrete was placed by 1-cubic-yard, buckets handled by floating cranes.

Small-diameter immersion vibrators, with flexible shaft drives, were used in conjunction with improvised form vibrators for the compaction of the concrete. The over-run of concrete in the entire job was approximately 8 per cent and that was due to yielding of the forms.

Formwork

All forms were of wood and lined with $1\frac{1}{4}$-inch tongued and grooved cypress. Cell forms were supported on tapered concrete blocks for the first or base pour; leakage around these proved to be negligible when a water test was made later.

Construction Joints

The surface of the construction joint of the first pour was kept fairly even and well compacted. Curing of this joint was assisted by completely filling the cells with water and allowing it to flow over the joint for as long as possible. Immediately before the erection of the second-pour steel, the whole joint was sand blasted to expose the aggregate over the entire area of the joint, and before pouring concrete the joint was well coated with about 1 inch of mortar worked into the joint with trowels.

Curing

The first and second pours were cured by filling the cells with water and by hosing. Damp sand was used for curing the deck pours.

Testing and Repair of Imperfections

All testing of hulls was carried out by merely filling the cells with water to a height not exceeding the height to which the vessel was to be submerged in use. Any patches showing damp on the outside were cut out and patched up by guniting with the cement gun. The resulting patches were invariably sound, the gunite so produced being perfectly watertight.

Painting

The outer walls were given four spray coats of polyplastic (polyvinyl chloride), the final two coats having an admixture of aluminium powder. The coats were applied as follows, with a minimum of 24 hours drying between coats:—

1st coat : 50 per cent polyplastic clear plus 50 per cent thinners
2nd coat : 75 ,, ,, ,, ,, ,, 25 ,, ,, ,,
3rd coat : polyplastic, plus aluminium powder
4th coat : ,, ,, ,, ,,

Two additional coats of polyplastic were applied in a band 12 inches wide across the construction joint. The underside of the pontoon was given one coat of clear polyplastic with 25 per cent thinners and two coats straight with aluminium powder added. After launching, the inside of the outer walls and the floor slab were given two coats of polyplastic (one clear, one aluminium).

Launching

As a suitable high-and-low tide sequence was available only at night at the particular date of launching, the operation was carried out under floodlight. Four 1,500-watt floodlights were provided ashore, and two portable generating sets supplied power for two 500-watt floodlights on the winch barges.

The pontoon was launched without the top deck, and in such a condition was calculated to weigh 480 tons. Six double strings of preassembled Army bridging pontoons, four with 32 units and two with 36 units comprising each double string, were used to lift the vessel on a rising tide. These Army pontoons were assembled on slipways and fitted with 1-inch air-lines at the top and 2-inch water-lines with control valves at the bottom. Oregon pine baulks, 12 inches square, were arranged to lift beneath the bulkheads and walls.

The pontoon strings were inserted beneath the concrete pontoon at a suitable low tide, and held in a central position in each bay ; marks were painted on all sets and on the concrete vessel facilitating this operation.

The vessel lifted on the rising tide when the water level had reached the 4-foot mark as predicted, leaving the Army pontoons with 9 inches of freeboard, which also confirmed calculations.

Floating-off from the staging, with the aid of two hand winches located on timber barges moored seaward of the staging, took only a few minutes and was accomplished without mishap. The whole assembly was moored clear of the staging to allow patching and painting of portions of the underside of the pontoon, which had not been accessible when the vessel was resting on the staging.

The scuttling or actual launching operation was performed the following morning by opening the water valves attached to the Army pontoons and controlling the rate of sinking by opening or closing air cocks located on an air manifold attached to the concrete pontoon. When the steel pontoons had settled on the bottom, the concrete pontoon was withdrawn and moored alongside the jetty ; the Army pontoons were raised by inflating with compressed air.

Fenders and Fittings

After launching, the pontoon was completed by adding the concrete deck, manhole frames, and other ironwork. Belting timbers were also fitted and bolted into position. The vessel was then towed to another location to have a covering steel-framed shelter erected and the shore ramp fitted before installation in its final position.

The Paper is accompanied by fourteen photographs and four sheets of drawings and diagrams, from some of which the Figures in the text have been prepared.

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